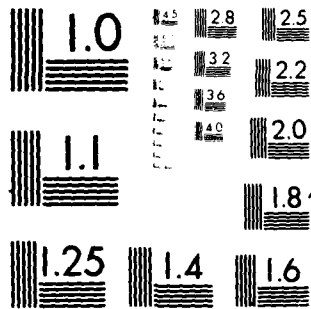
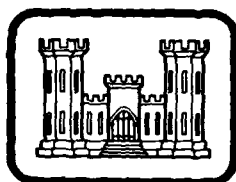


AD-A096 822 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/2
STABILITY OF RUBBLE-MOUND BREAKWATER, MAALAEA HARBOR, MAUI, HAW--ETC(U)
JAN 81 R D CARVER, D G MARKLE
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MISCELLANEOUS PAPER HL-81-1

STABILITY OF RUBBLE-MOUND BREAKWATER MAALAEA HARBOR, MAUI, HAWAII

Hydraulic Model Investigation

by

Robert D. Carver, Dennis G. Markle

Hydraulics Laboratory

U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

January 1981

Final Report

Approved For Public Release; Distribution Unlimited

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Prepared for U. S. Army Engineer Division, Pacific Ocean
Fort Shafter, Hawaii 96858

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REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER Miscellaneous Paper HL-81-1 ✓	2. GOVT ACCESSION NO. AD-A096822	3. RECIPIENT'S CATALOG NUMBER 9
4. TITLE (and Subtitle) STABILITY OF RUBBLE-MOUND BREAKWATER, MAALAEA HARBOR, MAUI, HAWAII, Hydraulic Model Investigation	5. TYPE OF REPORT & PERIOD COVERED Final report. Apr-Jun 80	
7. AUTHOR(s) Robert D. Carver Dennis G. Markle	6. PERFORMING ORG. REPORT NUMBER	
9. PERFORMING ORGANIZATION NAME AND ADDRESS U. S. Army Engineer Waterways Experiment Station Hydraulics Laboratory P. O. Box 631, Vicksburg, Miss. 39180	8. CONTRACT OR GRANT NUMBER(s)	
11. CONTROLLING OFFICE NAME AND ADDRESS U. S. Army Engineer Division, South Pacific Building 230 Fort Shafter, Hawaii 96858	10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS	
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office) 12 55	12. REPORT DATE January 1981	
	13. NUMBER OF PAGES 50	
	15. SECURITY CLASS. (of this report) Unclassified	
	15a. DECLASSIFICATION/DOWNGRADING SCHEDULE	
16. DISTRIBUTION STATEMENT (of this Report) Approved for public release; distribution unlimited.		
17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)		
18. SUPPLEMENTARY NOTES		
19. KEY WORDS (Continue on reverse side if necessary and identify by block number) Breakers (Water waves) Breakwaters Hydraulic models Maalaea Harbor, Hawaii Rubble-mound breakwaters		
20. ABSTRACT (Continue on reverse side if necessary and identify by block number) - An undistorted-scale hydraulic model study was conducted to investigate the adequacy of three breakwater cross sections considered for use at Maalaea Harbor, Maui, Hawaii. Plan 1 was protected by 6-ton dolosse on both the sea side and beach side, whereas Plans 2 and 3 used 6-ton dolosse sea side and 7- to 10-ton stone beach side. Plans 1 and 2 used 1V- on 2H-armor slopes and Plan 3 used 1V- on 1.5H-armor slopes. Based on results of model tests, it was concluded that (Continued)		

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20. ABSTRACT (Continued).

Plans 1, 2, and 3 are stable designs for the maximum breaking wave heights that can be expected to occur for 12- to 16-sec waves at swl's of -1 and +4 ft mllw. Also, Plans 2 and 3 can withstand attack of 16-sec, 15.2-ft breaking waves at an swl of +6 ft mllw and 16-sec, 16.7-ft breaking waves at an swl of +8 ft mllw without experiencing significant damage.

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Preface

The model investigation reported herein was requested by the U. S. Army Engineer Division, Pacific Ocean (POD), in a letter to the U. S. Army Engineer Waterways Experiment Station (WES) dated 7 February 1980. The investigation was authorized by POD Intra-Army Order PODSP-CIV-80-21 dated 7 March 1980.

Model tests were conducted at WES during the period April through June 1980, under the general direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, and Dr. R. W. Whalin, Chief of the Wave Dynamics Division. Tests were conducted by Messrs. R. D. Carver and D. G. Markle, Research Hydraulic Engineers, and Mr. C. R. Herrington, Engineering Technician. Execution of this study and preparation of this report were performed by Messrs. Carver and Markle under the supervision of Mr. D. D. Davidson, Chief of the Wave Research Branch.

Liaison between POD and WES was maintained during the course of the investigation by telephone conversations and progress reports.

Commander and Director of WES during the conduct of the study and the preparation and publication of this report was COL Nelson P. Conover, CE. Technical Director was Mr. F. R. Brown.

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Conversion Factors, U. S. Customary to Metric (SI)
Units of Measurement

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
tons (2000 lb, mass)	907.1847	kilograms

STABILITY OF RUBBLE-MOUND BREAKWATER

MAALAEA HARBOR, MAUI, HAWAII

Hydraulic Model Investigation

Introduction

1. Maalaea Harbor is located on the west coast of Maui, one of the larger islands in the Hawaiian Island group (Plate 1). Pleasure boating, sport fishing, and commercial fishing have increased significantly during recent years and expansion of an existing small-boat harbor has been proposed. In order to provide wind wave and swell protection for the expanded harbor, it will be necessary to extend the existing south breakwater.

Purpose of Model Study

2. The original purpose of the model study was to experimentally investigate the adequacy of two breakwater sections proposed by the U. S. Army Engineer Division, Pacific Ocean (POD), for the south breakwater extension. Both sections used 1V- on 2H-armor slopes. One alternative was protected by 6-ton* dolosse on both the sea side and beach side, whereas the other employed 6-ton dolosse sea side and 7- to 10-ton stone beach side. Later, following completion of tests for the two original plans, it was decided to also investigate a third plan using 1V- on 1.5H-armor slopes in an attempt to reduce costs for construction of the breakwater.

Design of Model

3. Tests were conducted at an undistorted linear scale of 1:27.5, model to prototype. Scale selection was based on the size of

* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

model armor units available relative to the estimated size of prototype armor units required for stability, the elimination of stability scale effects* and capabilities of the available wave tank. Based on Froude's model law** and the linear scale of 1:27.5, the following model-to-prototype relations were derived. Dimensions are in terms of length (L) and time (T).

<u>Characteristics</u>	<u>Dimensions</u>	<u>Model:Prototype Scale Relations</u>
Length	L	$L_r = 1:27.5$
Area	L^2	$A_r = L_r^2 = 1:756$
Volume	L^3	$V_r = L_r^3 = 1:20,797$
Time	T	$T_r = L_r^{1/2} = 1:5.24$

4. The specific weight of water used in the model was assumed to be 62.4 pcf and that of seawater is 64.0 pcf. Also, specific weights of model breakwater construction materials were not the same as their prototype counterparts. These variables were related using the following transference equation:

$$\frac{(W_r)_m}{(W_r)_p} = \frac{(\gamma_r)_m}{(\gamma_r)_p} \left(\frac{L_m}{L_p} \right)^3 \left[\frac{(S_r)_p - 1}{(S_r)_m - 1} \right]^3$$

where

Subscripts m, p = model and prototype quantities, respectively

W_r = weight of an individual armor unit or stone, lb

γ_r = specific weight of an individual armor unit or stone, pcf

* R. Y. Hudson. 1975 (Jun). "Reliability of Rubble-Mound Breakwater Stability Models; Hydraulic Model Investigation," Miscellaneous Paper H-75-5, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

** J. C. Stevens et al. 1942. "Hydraulic Models," Manuals on Engineering Practice No. 25, American Society of Civil Engineers, New York, N. Y.

L_m/L_p = linear scale of the model

S_r = specific gravity of an individual armor unit or stone relative to the water in which the breakwater is constructed, i.e., $S_r = \gamma_r/\gamma_w$

γ_w = the specific weight of water, pcf

5. Crown protection for all plans will be provided by a cast-in-place concrete rib cage. It was assumed that the prototype rib cage will be stable; therefore, it was not necessary that the model rib cage be dynamically similar to the prototype. The model rib cage, constructed of Plexiglas, was geometrically similar to the prototype, thus ensuring proper reflection and dissipation of incident wave energy.

Test Equipment

6. Tests were conducted in a portion (100 ft long, 5 ft wide, and 3 ft deep) of an L-shaped concrete flume, which has overall dimensions of 250 ft long, 50 and 80 ft wide at the top and bottom of the L, respectively, and 4.5 ft deep. The flume layout is shown in Plate 2. A 44-ft (model) length of 1V-cn-50H slope, representative of the existing prototype sea bottom, was molded and test sections were installed 29 ft beachward of the slope's toe. The test facility is equipped with a flap-type wave generator, capable of producing sinusoidal waves of various periods and heights. Test waves of the required characteristics were generated by varying the frequency and amplitude of the plunger motion. Changes in water-surface elevation as a function of time were measured by electrical wave-height gages in the vicinity of where the toe of the test sections was to be placed and recorded on chart paper by an electrically operated oscillograph. Measurement of wave heights (generator calibration) without test sections in place simulated existing conditions.

Method of Constructing Test Sections

7. Model breakwater sections were constructed to reproduce, as closely as possible, results of the usual methods of constructing

prototype structures. Core material, dampened as it was dumped by bucket or shovel into the flume, was compacted with hand trowels to simulate natural consolidation resulting from wave action during construction of the prototype breakwater. Once the core material was in place, it was sprayed with a low-velocity water hose to ensure adequate compaction of the material. Underlayer stone was then added by shovel and smoothed to grade by hand or with trowels but was not packed in place. Armor units used in the cover layer were placed in a random manner, i.e., laid down in such a way that no intentional interlocking of the units was obtained. Model elevations were controlled with an engineer's level to a tolerance of ± 0.005 ft.

Description of Plans 1 and 2

8. Plan 1, shown in Plate 3 and Photos 1-3, was constructed to a crown elevation of +13.0 ft mean lower low water (mllw) and used armor slopes of 1V on 2H both sea side and beach side. Crown protection was provided by a concrete rib cage, underlain with one layer of 6- to 8-ton stone. Individual concrete ribs were 15 ft long, 6 ft high, and 3 ft wide connected by two 3.0- x 1.5- x 1.5-ft concrete spacers which were flush with the top of the ribs at one-third points along the 15-ft length. The ribs used a 6-ft spacing center-to-center and in the prototype the bottom of each rib will be doweled to the stone underlayer. Both seaward and beachward slope protection was provided by 6-ton dolosse. Core stone weights of 100 to 500 lb were used, and the armor and core were interfaced with 1- to 2-ton underlayer stone. Plan 2 (Plate 4 and Photos 4-6) was the same as Plan 1 except that 7- to 10-ton stone was used to armor the beach side of the breakwater.

Selection of Test Conditions

9. Based on anticipated prototype wave conditions, it was decided that stability tests should consider 12-, 14-, and 16-sec wave periods at still-water levels (swl's) of -1.0 and +4.0 ft mllw. Model

observations indicated that for the selected wave periods and swl's, the corresponding maximum breaking wave was always more severe than any lesser wave height. Observations of incident wave forms at the structure showed that the worst breaking waves (as a function of wave period) which could be made experimentally to attack the section for the selected conditions were as follows:

swl ft mllw	Wave Period sec	Worst Breaking Wave Height ft
-1.0	12.0	8.9
-1.0	14.0	9.2
-1.0	16.0	10.5
+4.0	12.0	12.5
+4.0	14.0	12.6
+4.0	16.0	13.5

Model observations also indicated that due to consistently larger breaking wave heights, the 16-sec period condition was more severe than either the 12- or 14-sec period conditions. Wave heights at the 12- and 14-sec periods appeared to be very similar in severity. It was decided that for the range of wave conditions considered, the stability response of the test sections could be adequately evaluated by subjecting the test structures to the following storm surge hydrograph:

Step	swl	Test Wave		Prototype	Wave Type
	ft mllw	Period, sec	Height, ft	Duration, hr	
	-1.0	14.0	4.6	0.33	Shakedown
1	-1.0	14.0	9.2	1.00	Worst breaking
2	-1.0	16.0	10.5	1.00	Worst breaking
3	+4.0	14.0	12.6	1.00	Worst breaking
4	+4.0	16.0	13.5	1.00	Worst breaking
5	-1.0	14.0	9.2	1.00	Worst breaking
6	-1.0	16.0	10.5	1.00	Worst breaking

The above hydrograph is graphically depicted in Plate 5.

Test Results for Design Wave Conditions

10. During testing of Plan 1 some intermittent minor rocking of

a few sea-side armor units was observed throughout the hydrograph and moderate wave overtopping during steps 3 and 4 caused minor rocking of a few beach-side armor units. Photos 7-9 show the test section after wave attack. A comparison of before- and after-test photographs shows that the final stabilized condition of the structure is virtually indistinguishable from its original appearance.

11. Plan 2 also exhibited an excellent stability response. Minor rocking of a few sea-side armor units was observed throughout the hydrograph; however, no displaced damage occurred. Even though moderate wave overtopping was present during steps 3 and 4, no damage was incurred by the 7- to 10-ton, beach-side armor stone. Photo 10 shows a 16-sec, 13.5-ft wave (step 4) impinging on the breakwater and Photo 11 shows the wave overtopping the structure. Photos 12-14 show the structure after completion of the hydrograph. Test results of Plans 1 and 2 were verified by a complete reconstruction and retesting.

Safety-Factor Tests of Plan 2

12. In designing rubble-mound breakwaters, or with any engineered structures, it is advantageous to determine what margin of safety is present in the selected designs. Consequently, at the conclusion of the repeat hydrograph test of Plan 2, it was decided to subject the structure to storm surges and wave heights in excess of the maximum design condition (16-sec, 13.5-ft waves at an swl of +4 ft mllw). A check of calibration data revealed that for the maximum design wave period of 16 sec, the wave generator was capable of producing depth-limited breaking waves for swl's up to +8 ft mllw. Therefore, even though POD realized swl's above the +4 ft mllw were extreme events, they requested Plan 2 be tested with 16-sec breaking waves at swl's of +6 and +8 ft mllw. Observations of incident wave forms at the structure showed that the worst breaking wave conditions which could be made experimentally to attack the section at the +6 and +8 ft swl's had heights of 15.2 and 16.7 ft, respectively.

13. Following completion of the repeat hydrograph test of Plan 2,

the water level was raised to +6 ft mllw and the structure was subjected to 16-sec, 15.2-ft waves. This condition produced significant wave overtopping, resulting in displacement of two beach-side armor units. In-place rocking of 1 to 2 percent of the sea-side armor units (total number of dolosse used on the sea side of the model cross section was approximately 300 units) was observed; however, no displaced damage was experienced. Photos 15-17 show the after-testing condition of the breakwater.

14. Upon completion of testing at the +6 ft swl, the water level was raised to +8 ft mllw and the test section was subjected to attack of 16-sec, 16.7-ft waves. This condition produced major wave overtopping. Water coming across the structure did not directly strike exposed armor, and no further dislocation of beach-side armor was observed. Several sea-side armor units between +8 and +13 ft mllw shifted in their original positions, and 2 to 3 percent of the dolosse were observed to rock in place; however, no displaced damage resulted. Photos 18-20 show the final stability condition of the structure.

15. Plan 2 was subjected to wave attack for 2 hr (prototype) at each swl investigated. This duration of wave attack allowed sufficient time for the structure to stabilize, i.e., time for all movement of armor to abate.

Rationale and Test Results for Plan 3

16. Safety-factor tests of Plan 2 demonstrated that the structure was able to withstand attack of waves well in excess of the maximum design wave height; therefore, it was decided to investigate alternative schemes that might substantially reduce the structure's cost without sacrificing its functional performance. Some of the factors that govern material volume and, therefore, initial construction costs are elevation and width of the crown, type and weight of armor, and slope on which the armor is placed. Based on discussions between POD and the U. S. Army Engineer Waterways Experiment Station (WES), it was decided that in this particular study the greatest volumetric reductions with the least

probable impact on functionality could probably be achieved by steepening both seaward and beachward armor slopes from 1V on 2H to 1V on 1.5H. Based on this rationale, Plan 3 (Plate 6 and Photos 21-23) was identical with Plan 2 except for the steeper slopes.

17. Stability test results for Plan 3 were favorable. Minor rocking of a few sea-side armor units was observed throughout the hydrograph; however, no displaced damage occurred. Beach-side armor was undamaged, even though substantial wave overtopping was present during steps 3 and 4. It should be noted that as opposed to Plans 1 and 2, there appeared to be an increase in wave overtopping during attack of 14-sec, 12.6-ft waves (step 3) and 16-sec, 13.5-ft waves (step 4). Photos 24-26 show the breakwater after completion of the hydrograph. Reconstruction and retesting of the model section verified results of the initial test.

18. Plan 3 also was tested with 16-sec, 15.2-ft breaking waves at an swl of +6 ft mllw and 16-sec, 16.7-ft breaking waves at an swl of +8 ft mllw. Attack of the 16-sec, 15.2-ft breaking waves produced in-place rocking of 2 to 3 percent of the sea-side armor units; however, no displaced damage was observed. Beach-side armor was stable, even though major wave overtopping was experienced. Photos 27-29 show the after-testing condition of the structure. Wave attack at the +8 ft swl caused several sea-side armor units above the swl to shift in their original positions and 3 to 4 percent of the dolosse rocked in place; however, no displaced damage occurred. Also, beach-side armor stone was again stable. Photos 30-32 show the final stability condition of the structure.

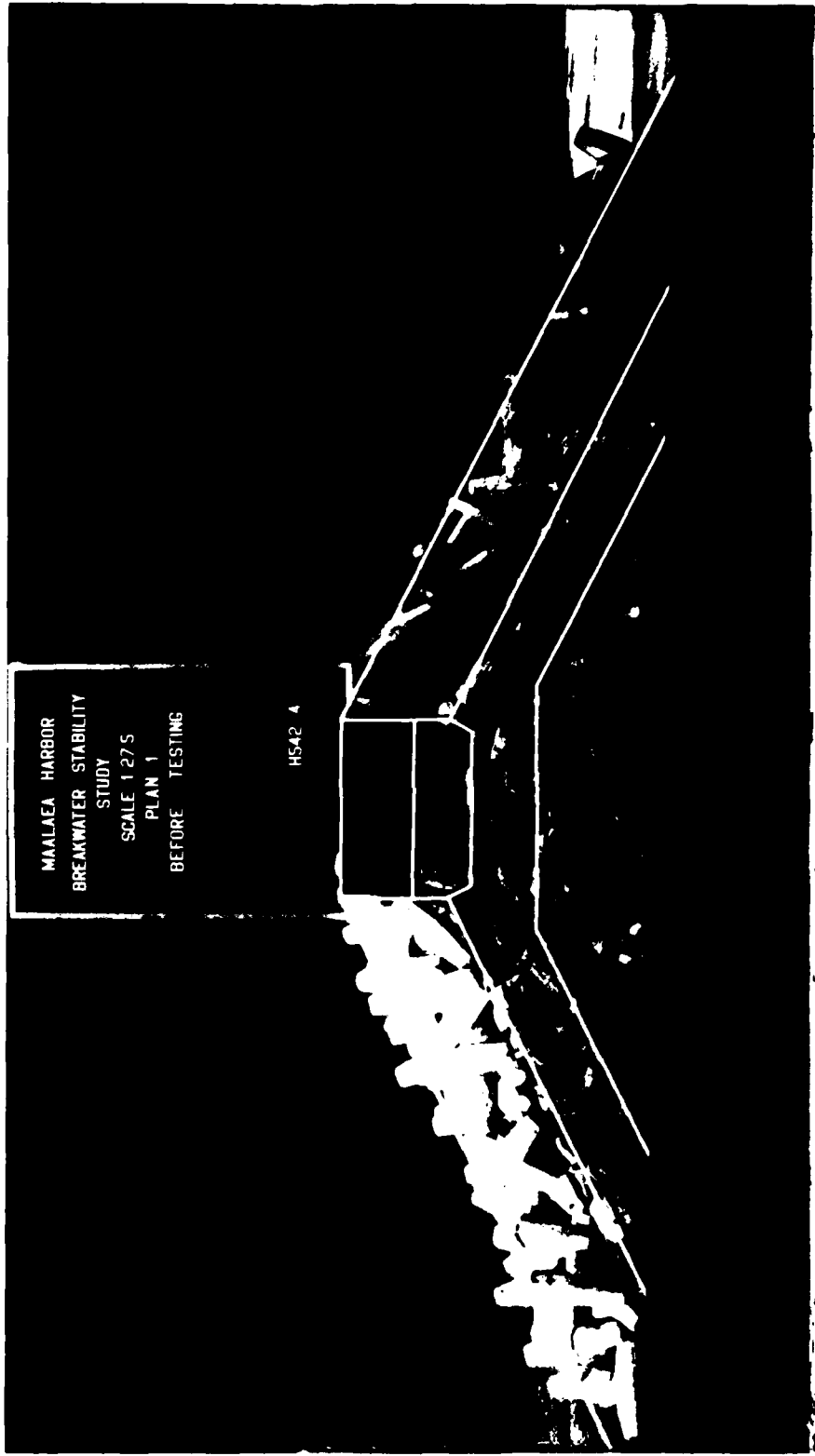
19. As with Plan 2, Plan 3 was subjected to wave attack for 2 hr (prototype) at each of the higher swl's (+6 ft mllw and +8 ft mllw). Again, this duration of wave attack allowed sufficient time for the structure to stabilize.

20. Stability test results of Plans 2 and 3 for the +6 and +8 ft swl's were very similar with slightly more in-place rocking of dolosse observed with Plan 3. Also, a moderate increase in wave overtopping and wave transmission was observed with Plan 3.

Conclusions

21. Based on the assumptions, tests, and results reported herein, it is concluded that:

- a. Plans 1, 2, and 3 are stable designs for the maximum breaking wave heights that can be expected to occur for 12- to 16-sec waves at swl's of -1 and +4 ft mllw.
- b. For safety-factor tests using extreme swl's, Plans 2 and 3 can withstand attack of 16-sec, 15.2-ft breaking waves at an swl of +6 ft mllw and 16-sec, 16.7-ft breaking waves at an swl of +8 ft mllw without experiencing significant damage.
- c. Stability test results of Plans 2 and 3 are very similar with slightly more in-place rocking of dolosse observed at the +6 and +8 ft swl's for Plan 3.
- d. As opposed to Plan 2, Plan 3 exhibited increased wave overtopping and wave transmission at the +4, +6, and +8 ft swl's.

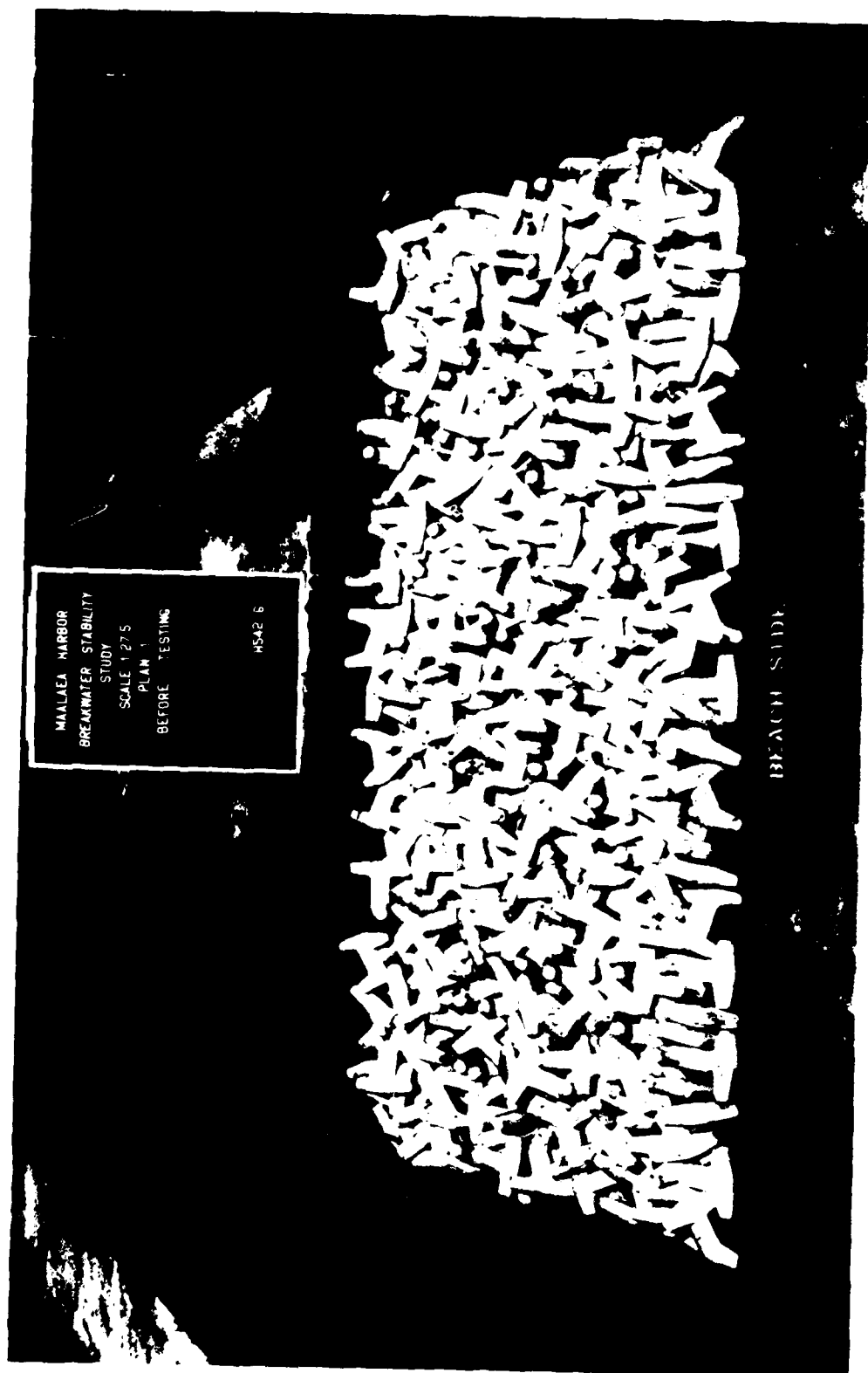


MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:275
PLAN 1
BEFORE TESTING

HS42 4

1. This is a plan view of the breakwater structure shown in the photograph above. The breakwater is 1,275 feet long and 10 feet wide. The building at the end of the breakwater is 10 feet by 10 feet. The water is 10 feet deep. The shoreline is 10 feet from the breakwater.



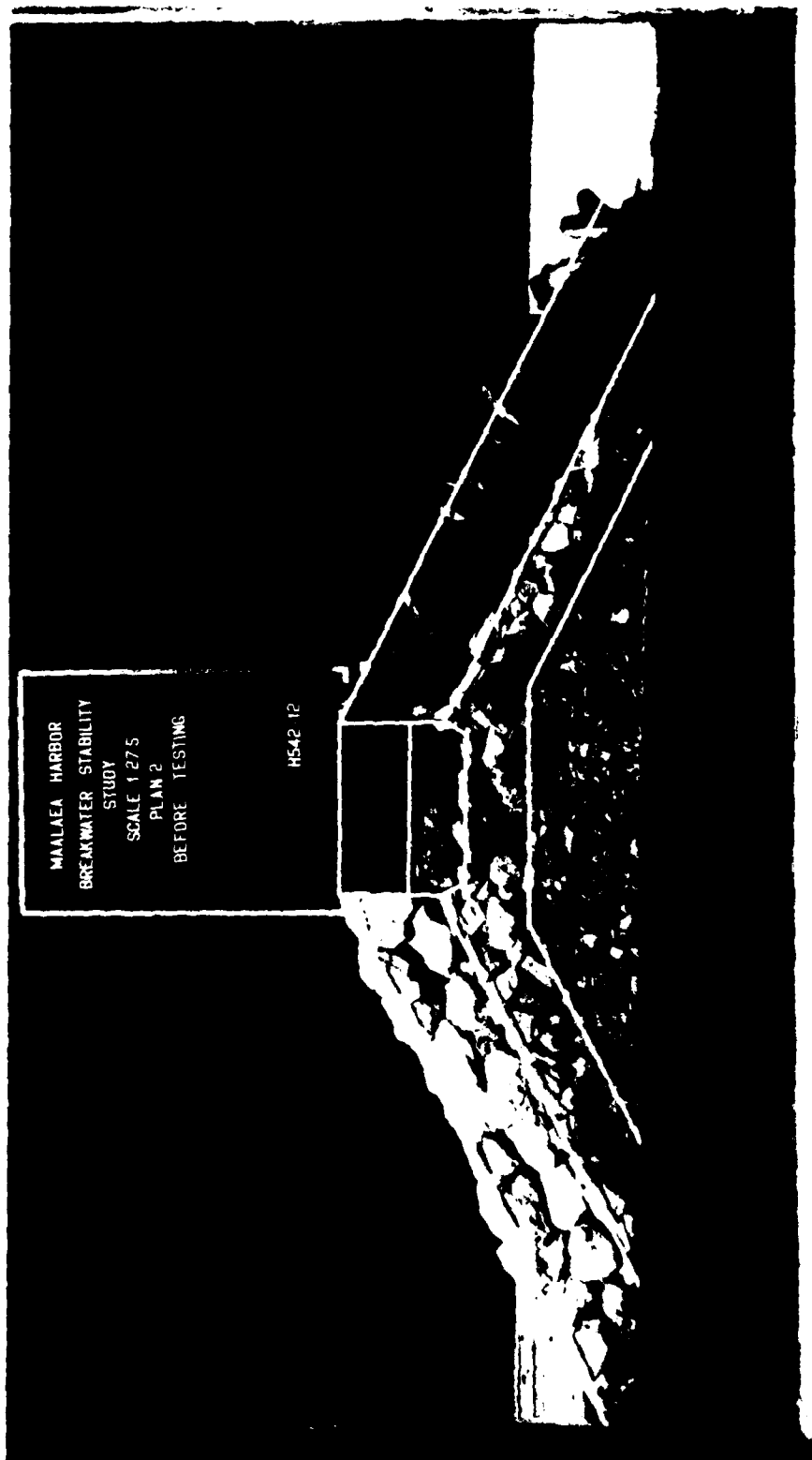


MALAE HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:275
PLAN 1
BEFORE TESTING

MS42 6

BEACH SIDE

Photo 3. Beach-side view of Plan 1 before wave attack



MAALAE HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:275
PLAN 2
BEFORE TESTING

H542 12

FIGURE 1. Aerial view of breakwater before testing.

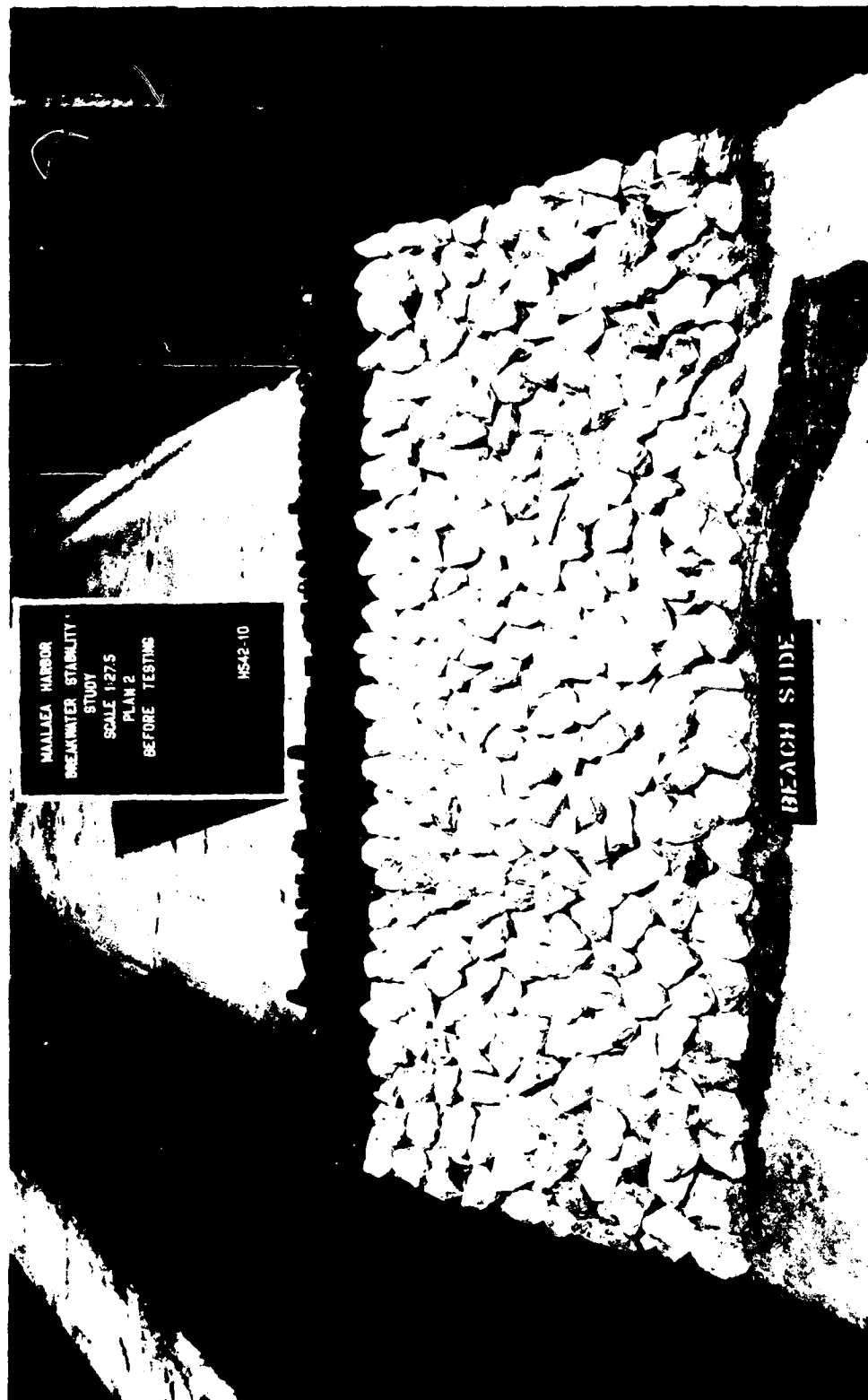


MALAYA HARBOR
BULKHEAD STABILITY
STUDY
SCALE 1/27.5
PLAN 2
BEFORE TESTING

HS42-11

SEA SIDE

400/194



MALLALA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 2
BEFORE TESTING

HS42-10

BEACH SIDE

Photo 6. Beach-slip view of Plan 2 before wave attack

MAALAE HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 1
AFTER TESTING

HS42-9



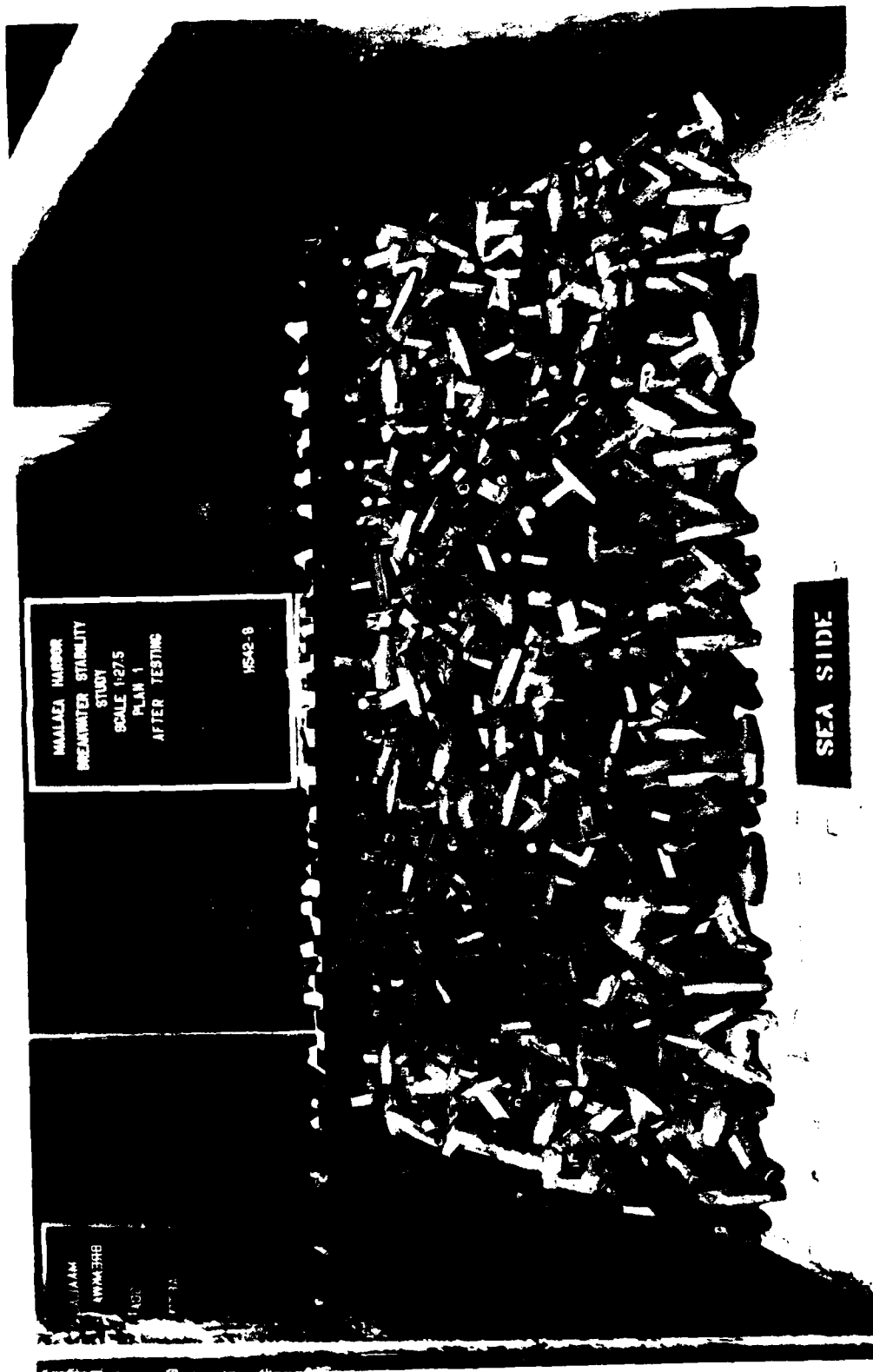
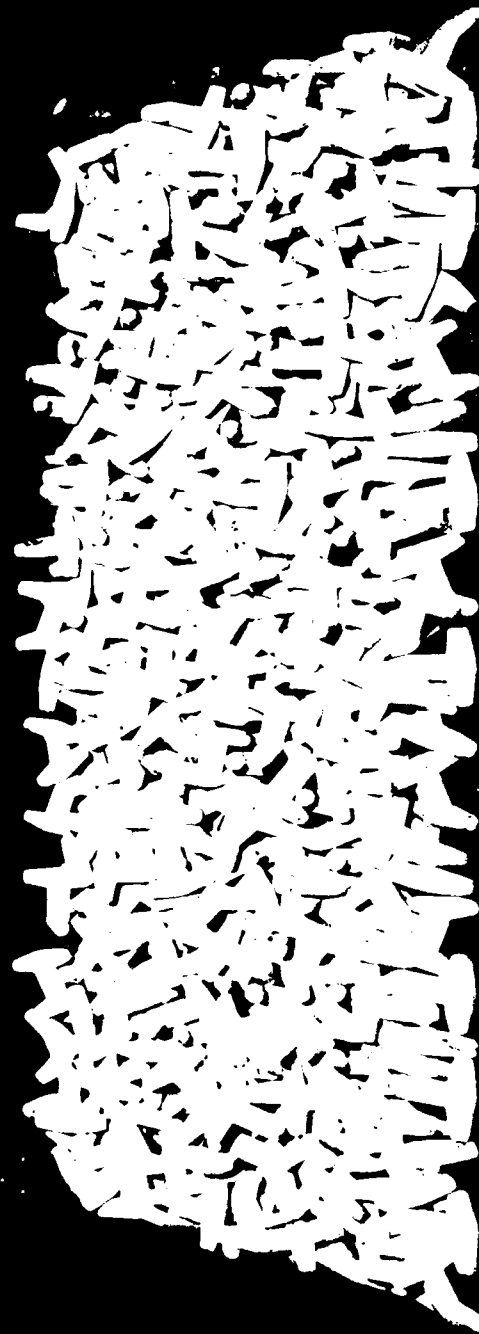


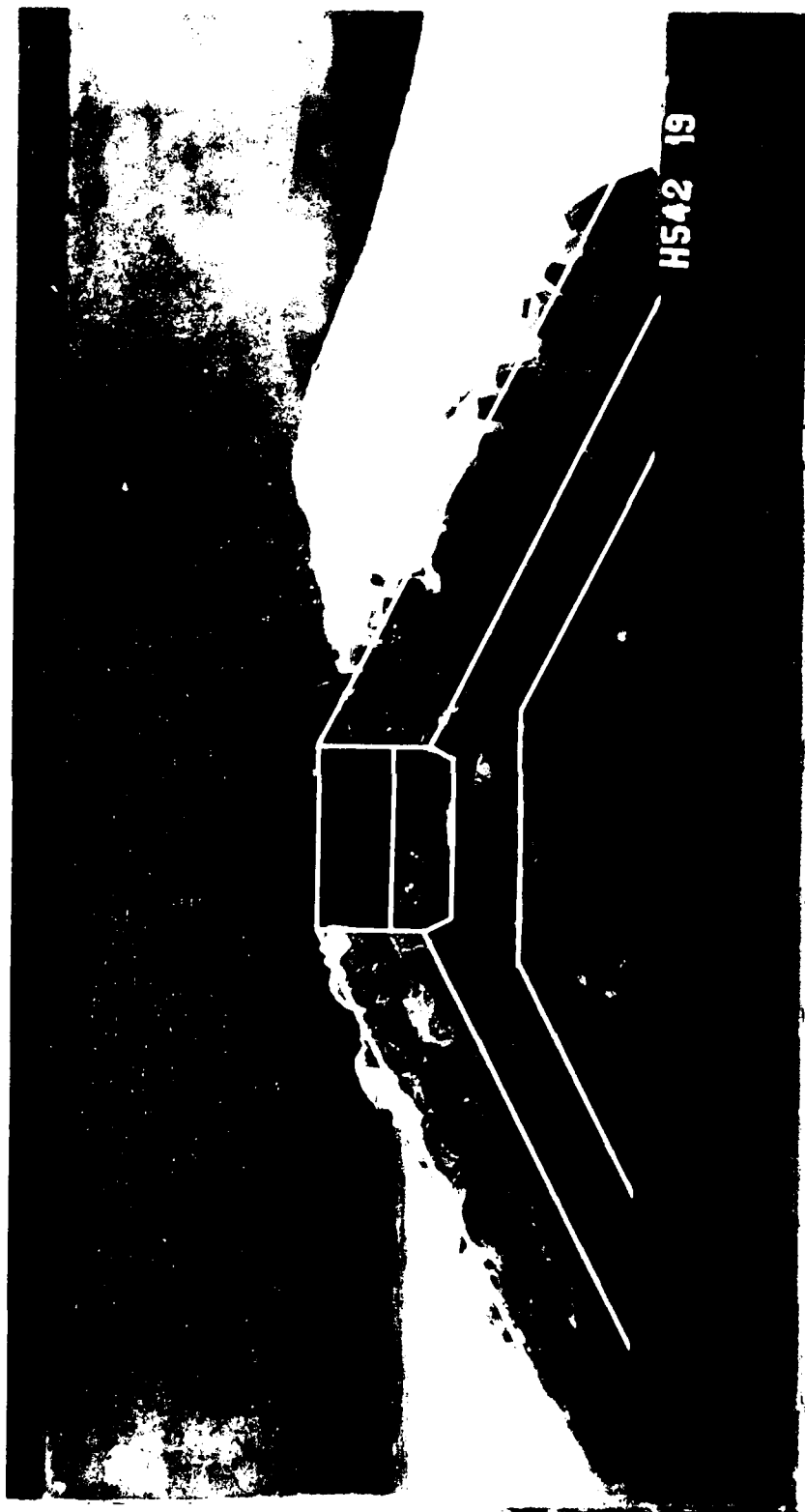
Photo 4. General view of Plan 1 after completion of submerged pipeline work.

MALIEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 1
AFTER TESTING

HS42-7

BEACH SIDE





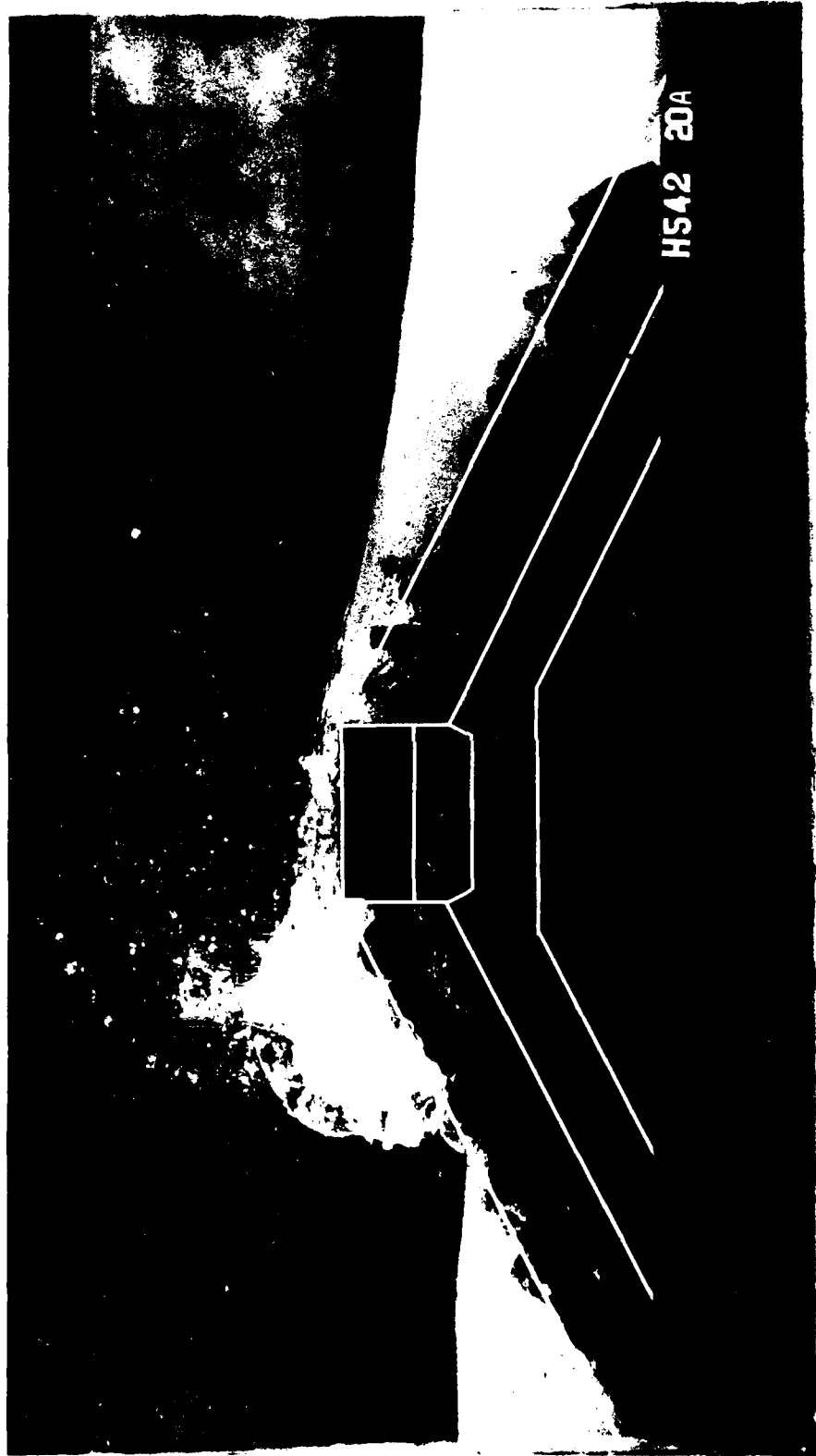


Photo 11. Aerial view of Runway 20A, H542 20A.

MAALAE HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:275
PLAN 2
AFTER TESTING

HS42 18

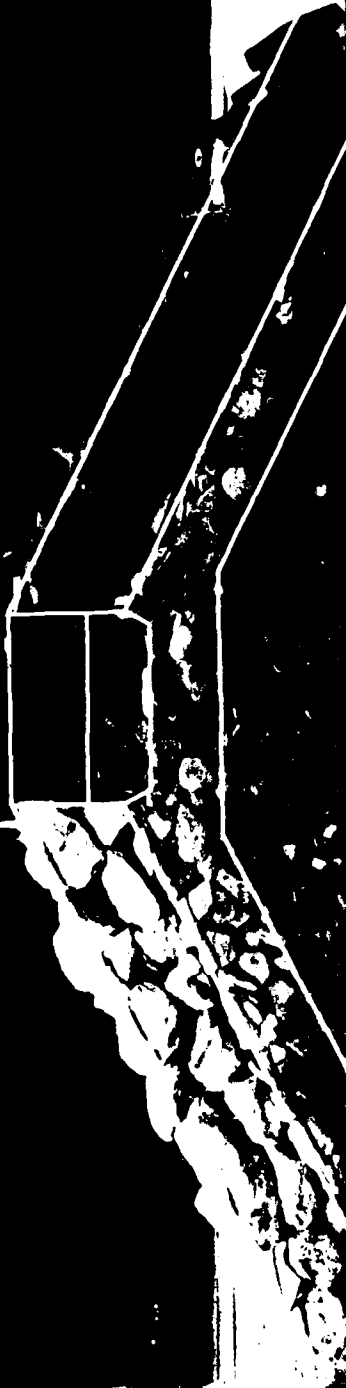
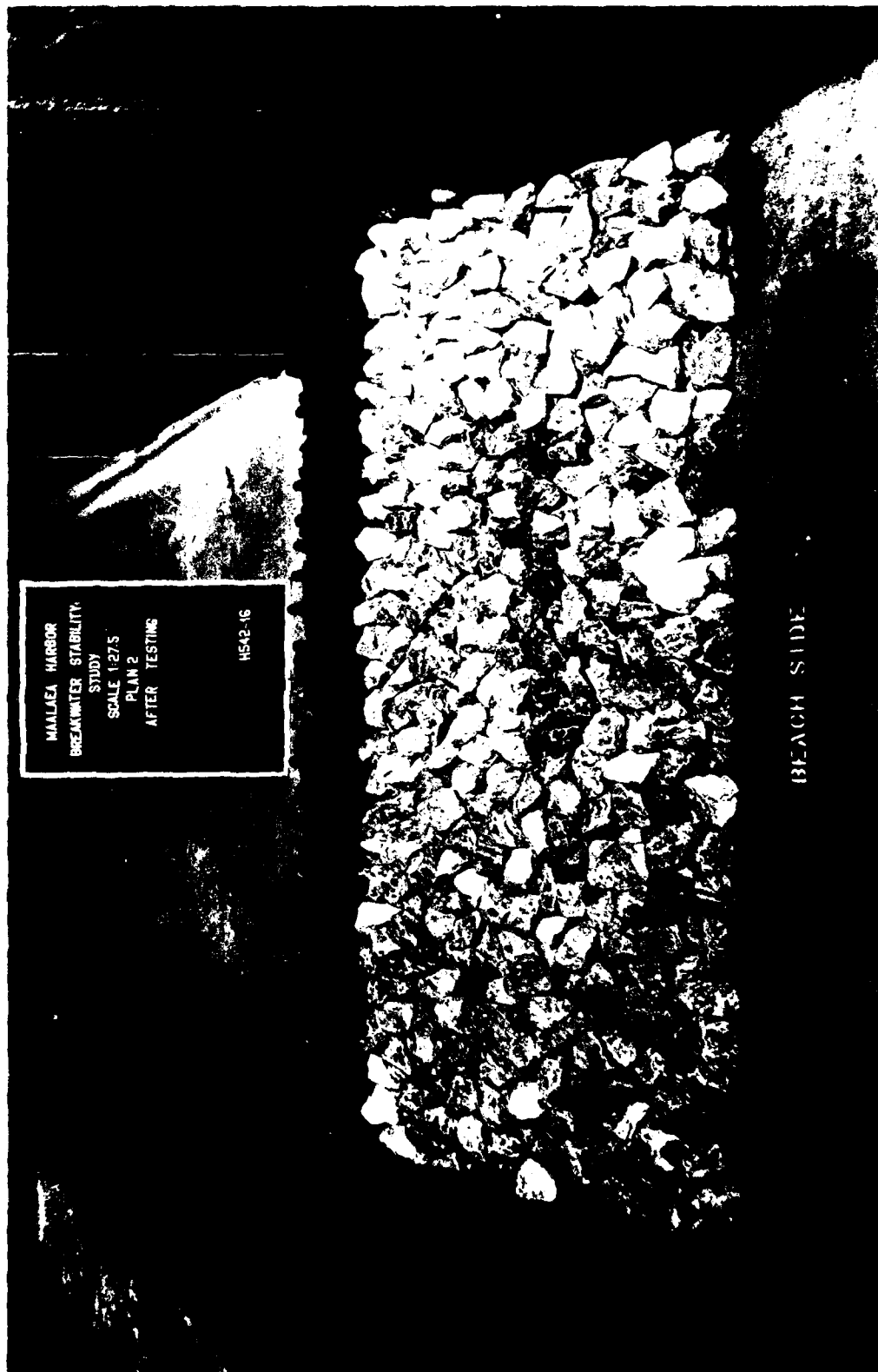


Figure 18. The view of Plan 2 after completion of storm-surge hydrograph.



Photo 14. Aerial view of physical model of breakwater stability study.

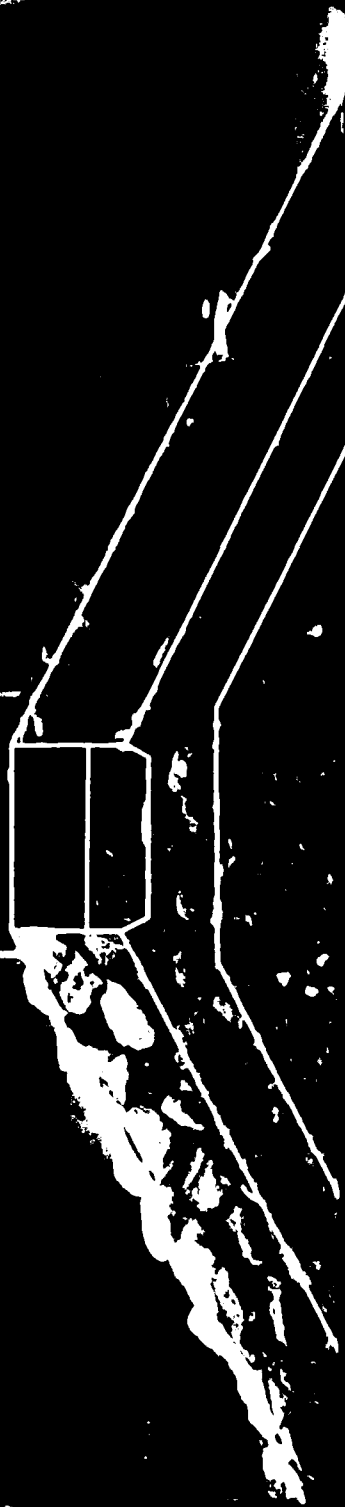


MAALIEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:275
PLAN 2
AFTER TESTING
HS42-16

BEACH SIDE

MAALAE HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1/27.5
PLAN 2
AFTER TESTING

HS42-23





MAALAE HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 2
AFTER TESTING

HS42-22

SEA SIDE

Figure 1. Breakwater stability study, after testing.

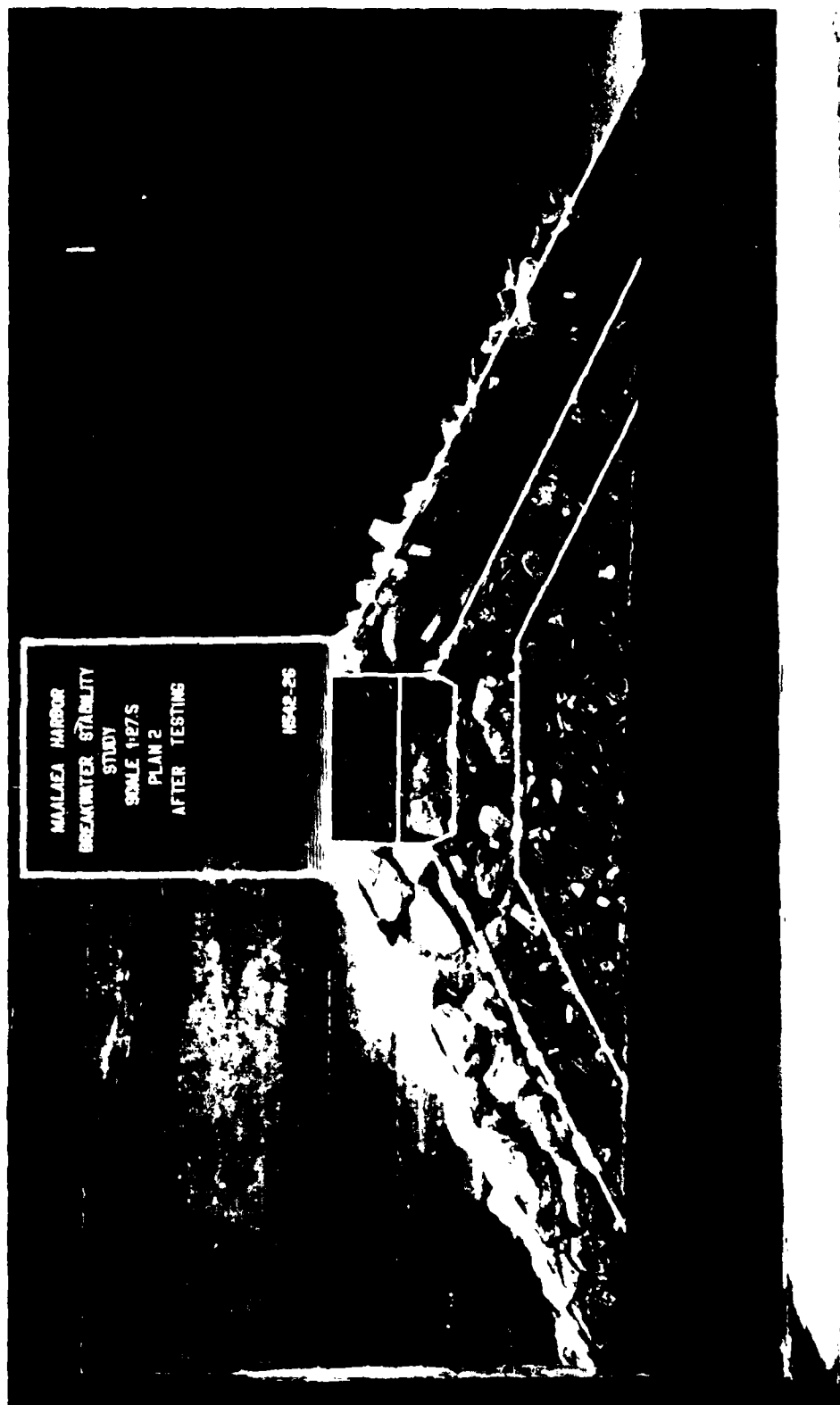
MALAJA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 2
AFTER TESTING

HS42-21



BEACH SIDE

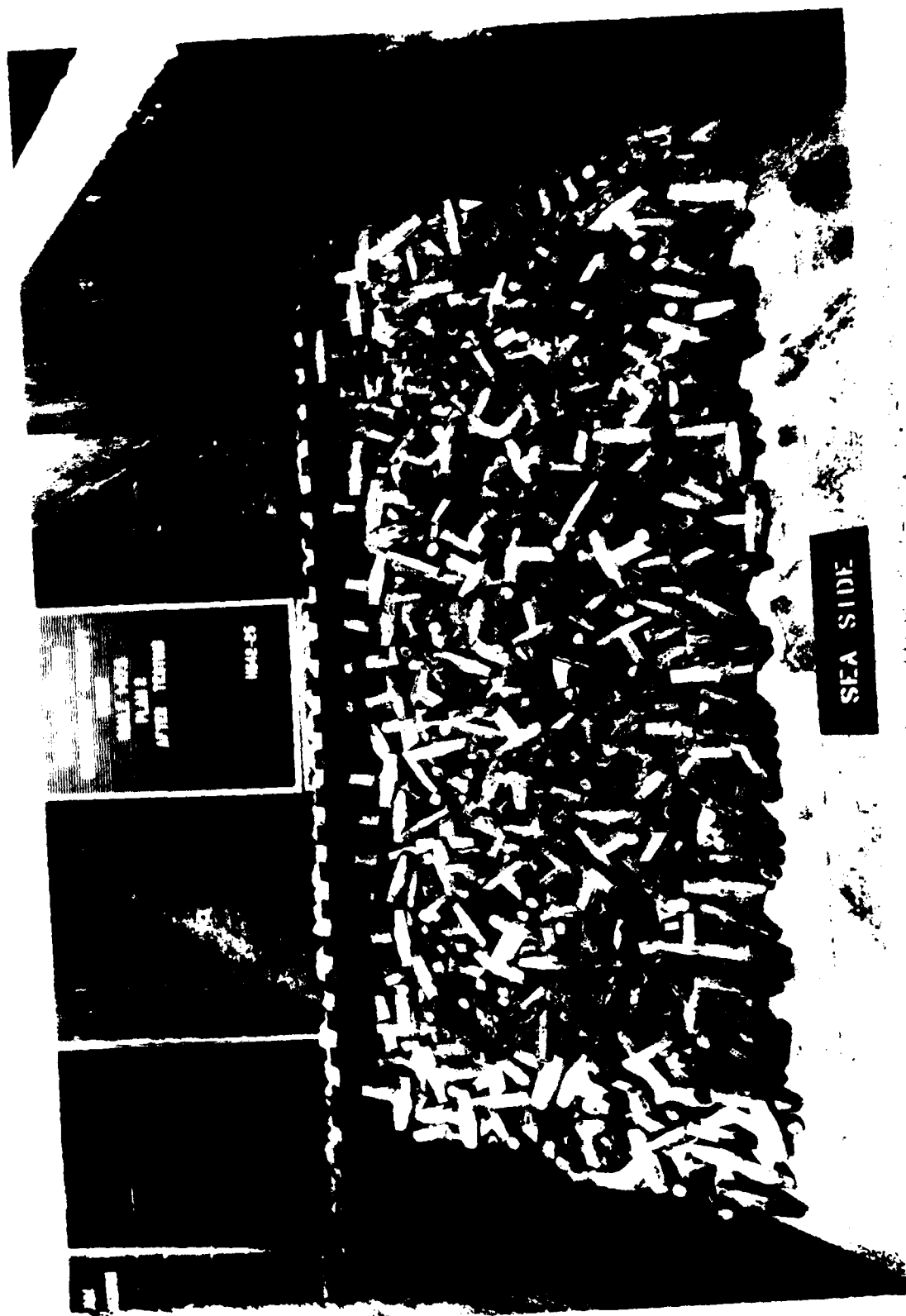
Photograph taken by the U.S. Army Corps of Engineers, 1962-63. Looking westward.



MALAE HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 2
AFTER TESTING

HS-42-25

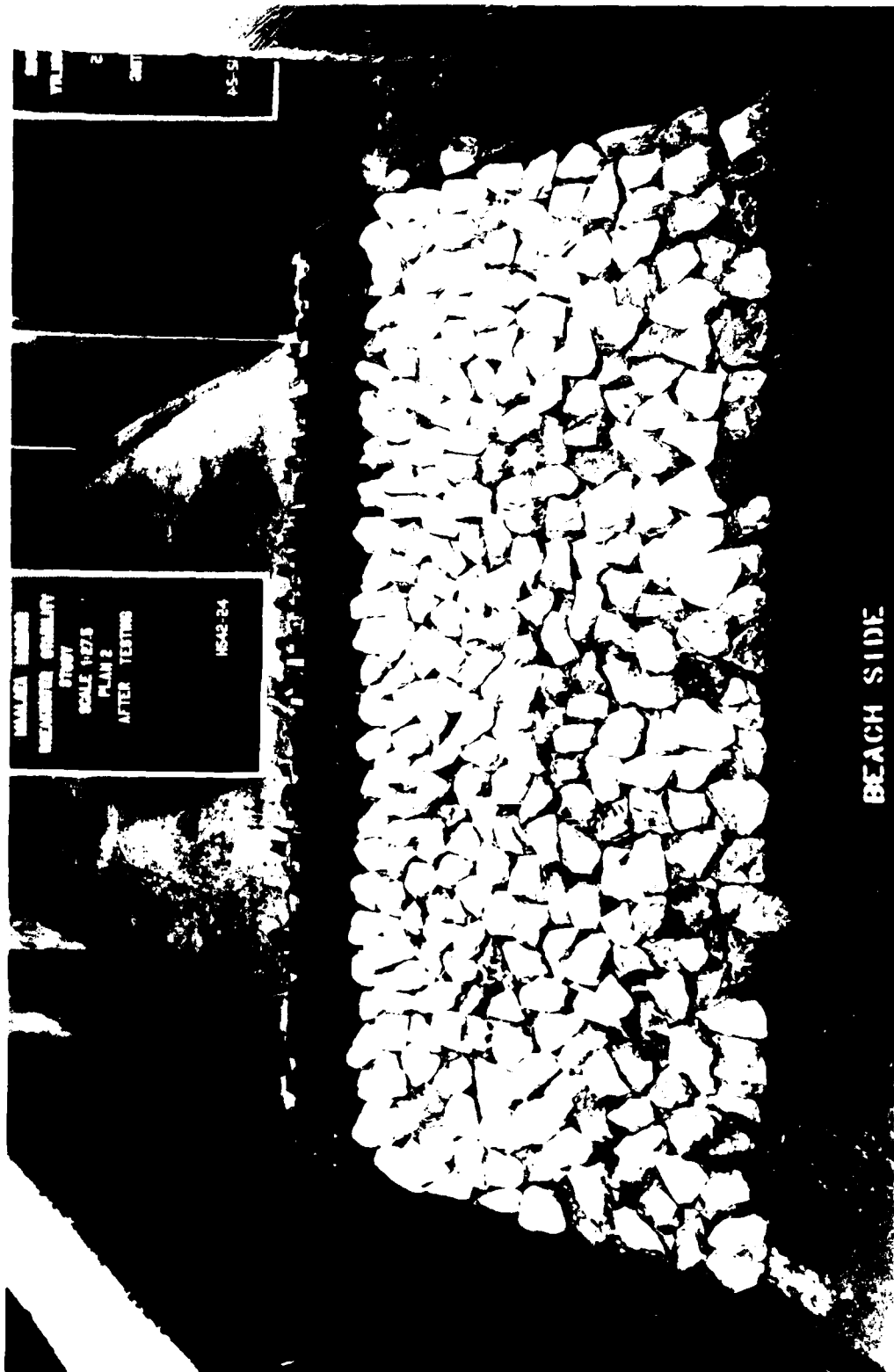
Photo 10. Side view of Plan 2 after study of 10-10-55, 10-11-55, 10-12-55, 11-1-56, 11-2-56, 11-3-56, 11-4-56, 11-5-56, 11-6-56, 11-7-56, 11-8-56, 11-9-56, 11-10-56, 11-11-56, 11-12-56, 12-1-57, 12-2-57, 12-3-57, 12-4-57, 12-5-57, 12-6-57, 12-7-57, 12-8-57, 12-9-57, 12-10-57, 12-11-57, 12-12-57, 1-1-58, 1-2-58, 1-3-58, 1-4-58, 1-5-58, 1-6-58, 1-7-58, 1-8-58, 1-9-58, 1-10-58, 1-11-58, 1-12-58, 2-1-59, 2-2-59, 2-3-59, 2-4-59, 2-5-59, 2-6-59, 2-7-59, 2-8-59, 2-9-59, 2-10-59, 2-11-59, 2-12-59, 3-1-60, 3-2-60, 3-3-60, 3-4-60, 3-5-60, 3-6-60, 3-7-60, 3-8-60, 3-9-60, 3-10-60, 3-11-60, 3-12-60, 4-1-61, 4-2-61, 4-3-61, 4-4-61, 4-5-61, 4-6-61, 4-7-61, 4-8-61, 4-9-61, 4-10-61, 4-11-61, 4-12-61, 5-1-62, 5-2-62, 5-3-62, 5-4-62, 5-5-62, 5-6-62, 5-7-62, 5-8-62, 5-9-62, 5-10-62, 5-11-62, 5-12-62, 6-1-63, 6-2-63, 6-3-63, 6-4-63, 6-5-63, 6-6-63, 6-7-63, 6-8-63, 6-9-63, 6-10-63, 6-11-63, 6-12-63, 7-1-64, 7-2-64, 7-3-64, 7-4-64, 7-5-64, 7-6-64, 7-7-64, 7-8-64, 7-9-64, 7-10-64, 7-11-64, 7-12-64, 8-1-65, 8-2-65, 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SEA SIDE

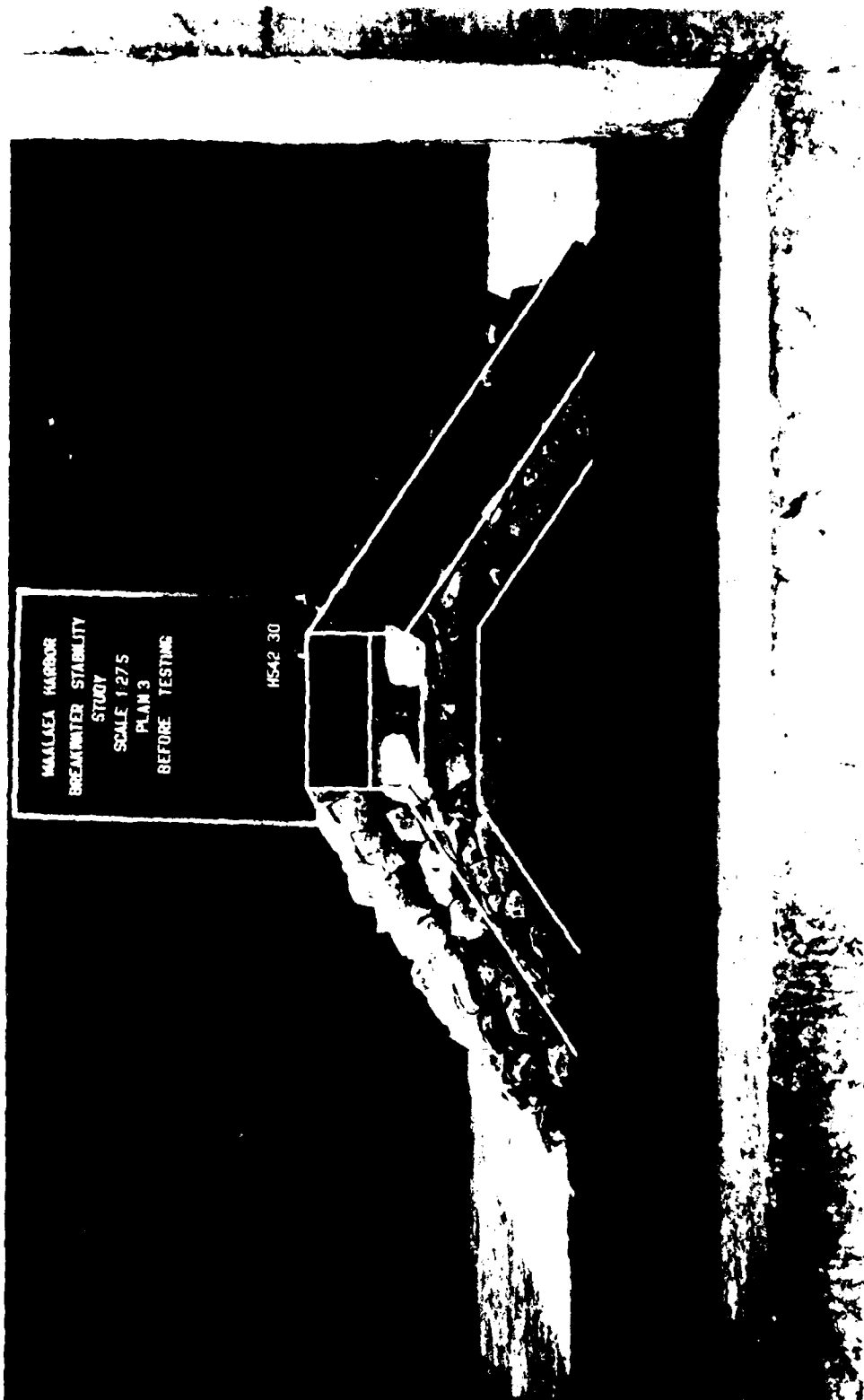
WALL 1402
PLAN 2
AFTER TESTING

1042-25



BEACH SIDE

Figure 1. Completed view of (1) and (2) after attack of 15-000, 16.7-04 breaking wave of 40
and 10 ft respectively.



MAALAE HARBOR
BREAKWATER STABILITY

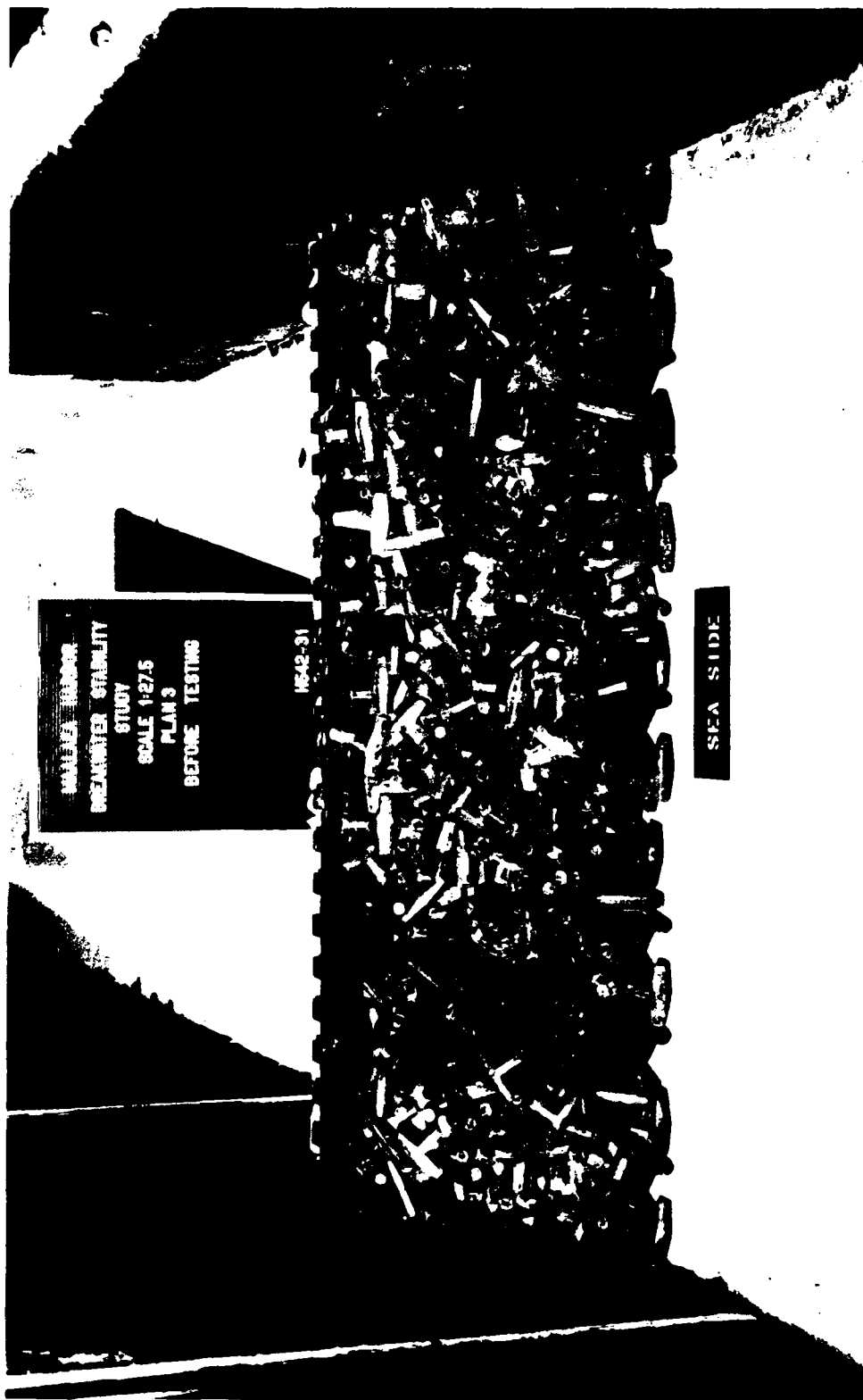
STUDY

SCALE 1/275

PLAN 3

BEFORE TESTING

HS42 30



WALLER HEDDER

DRAINAGE STABILITY

STUDY

SCALE 1:275

PLAN 3

BEFORE TESTING

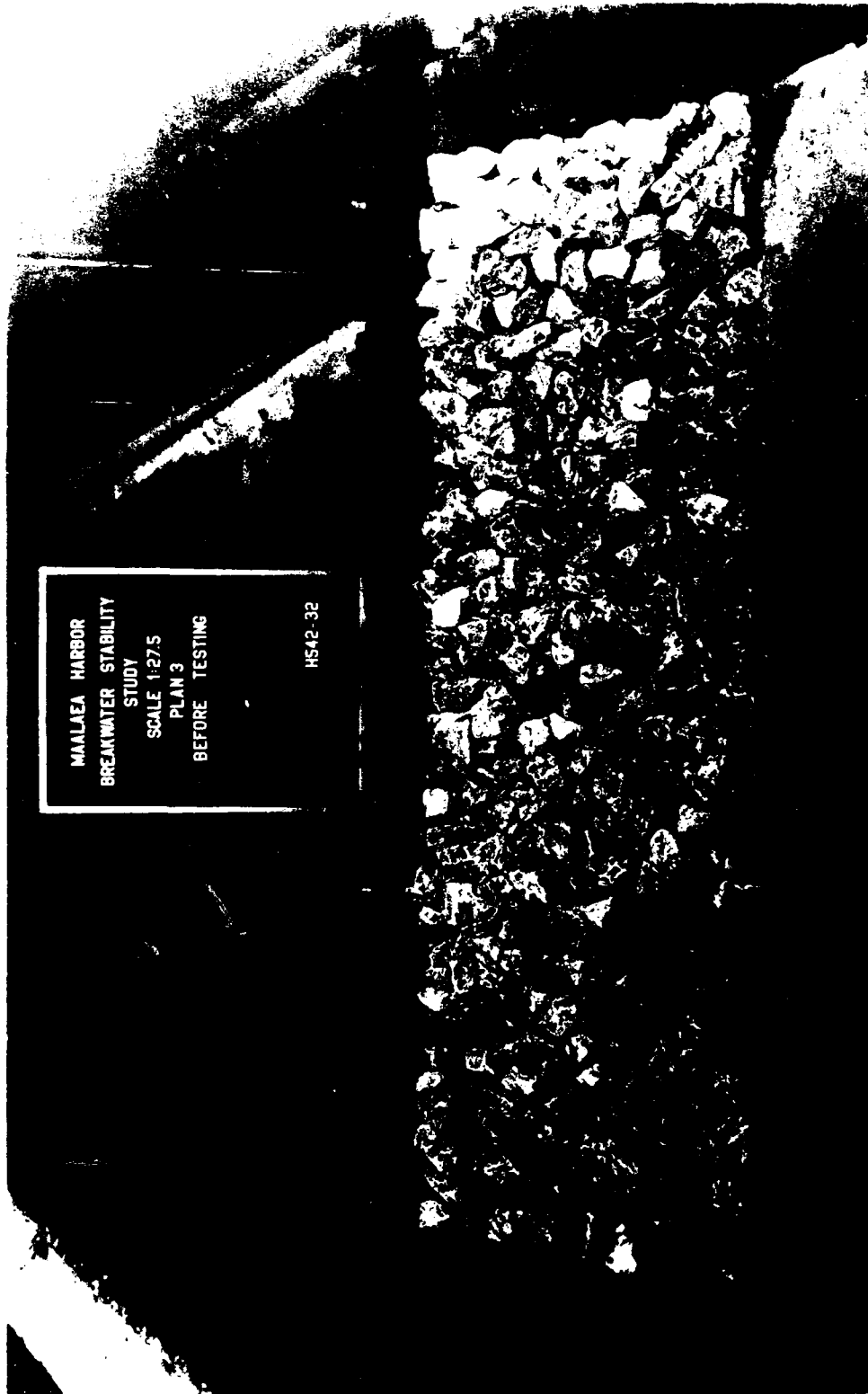
1642-31

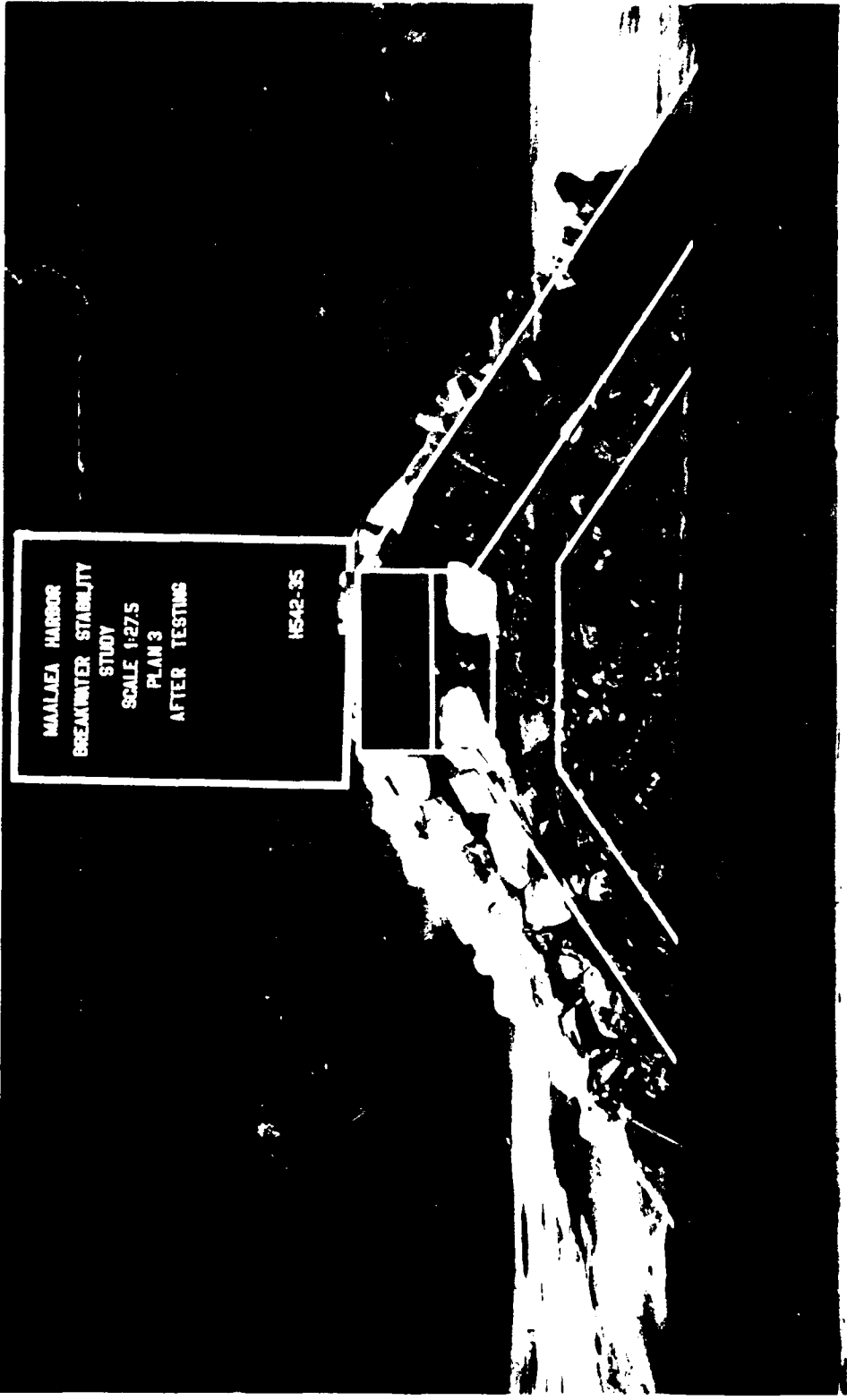
SEA SIDE

1642-31

MAALAE HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 3
BEFORE TESTING

HS42-32





MAALAE HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 3
AFTER TESTING

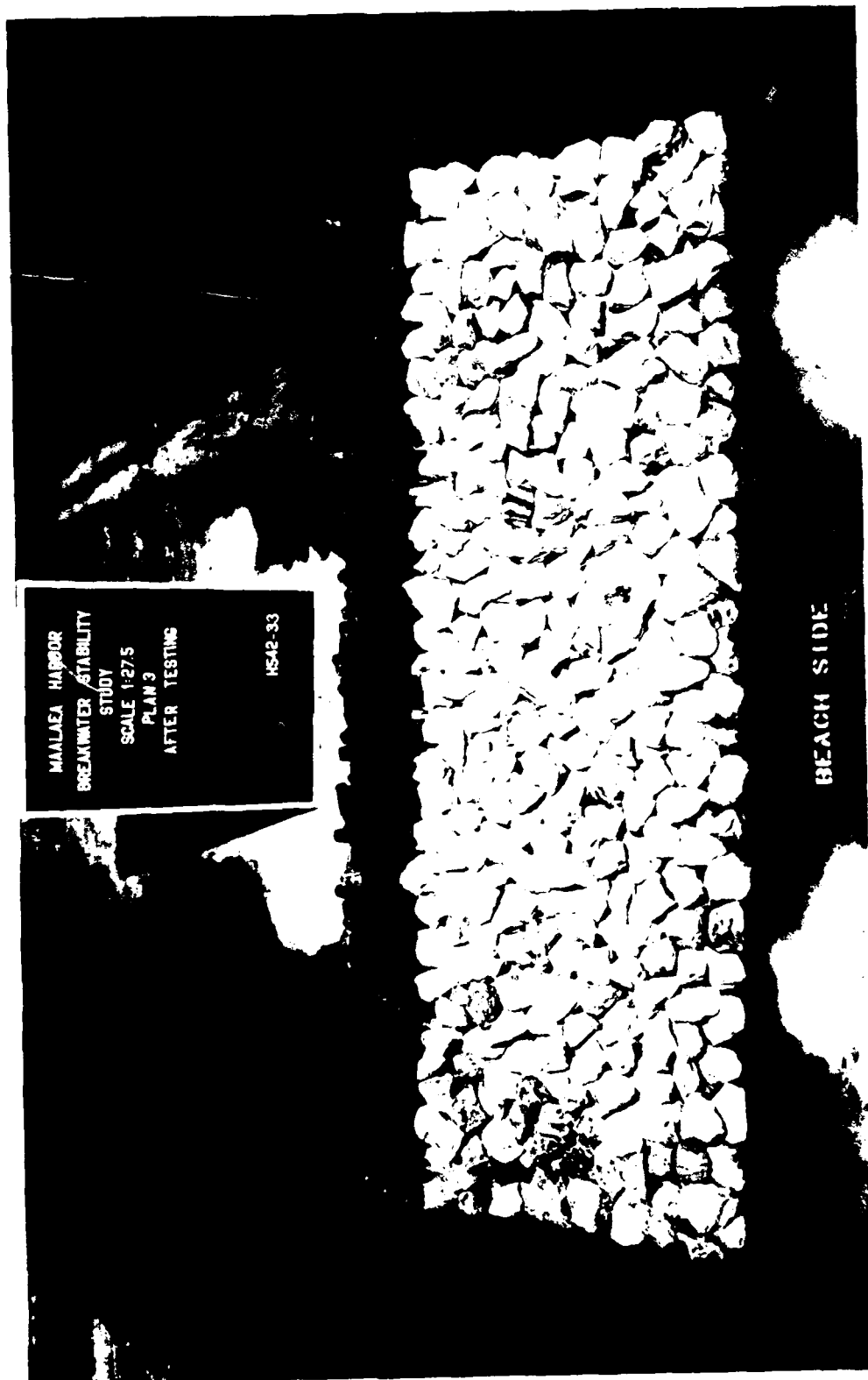
1642-35



MALALA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 3
AFTER TESTING

HS-12-34

SEA SIDE



MAALAE HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 3
AFTER TESTING

HS42-33

BEACH SIDE

Photo 29. Breakwater rubble pile, after testing.

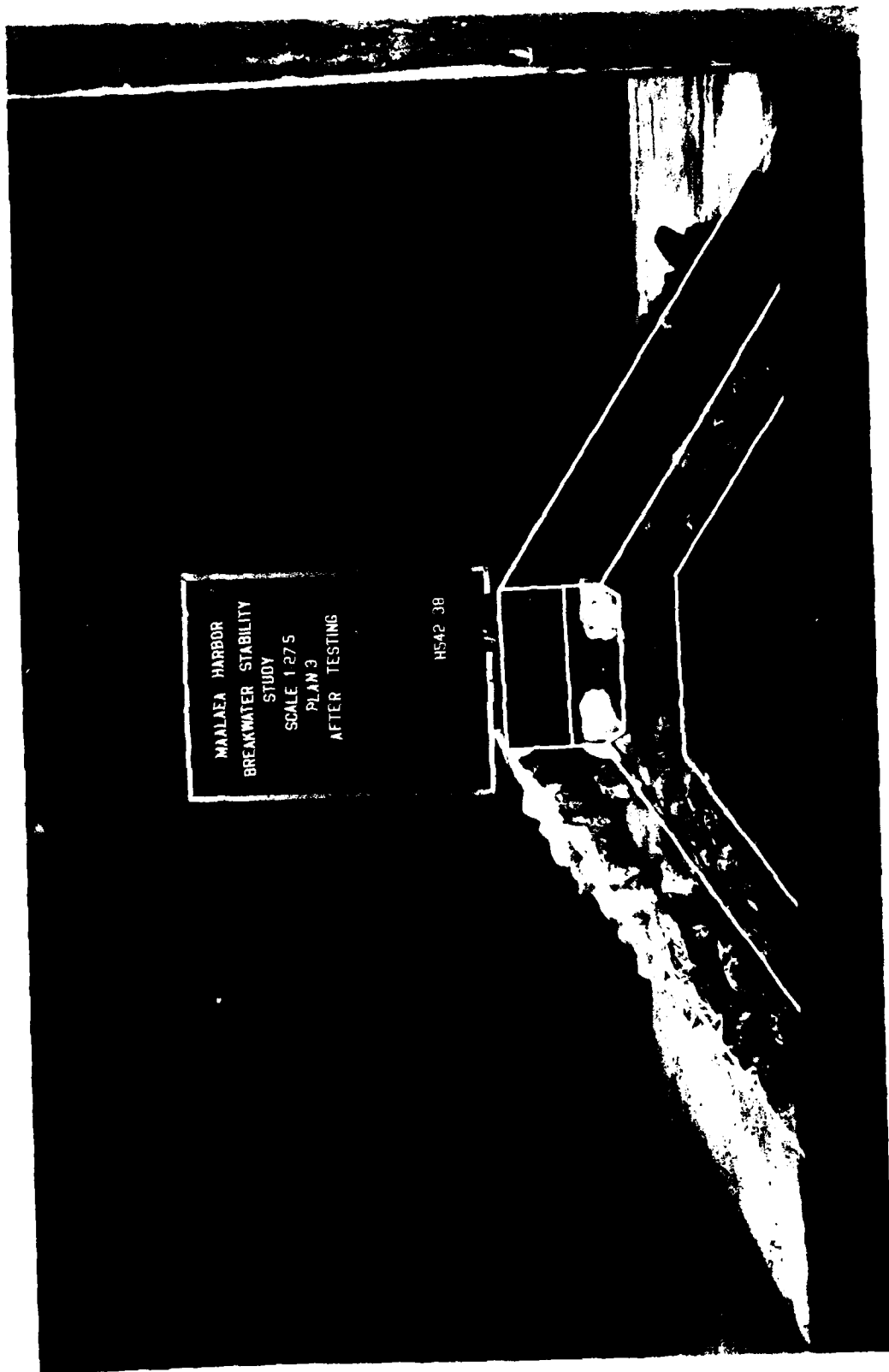


Photo 27. Side view of Plan 3 after attack of 100-ton projectile, 100 ft. from pier.

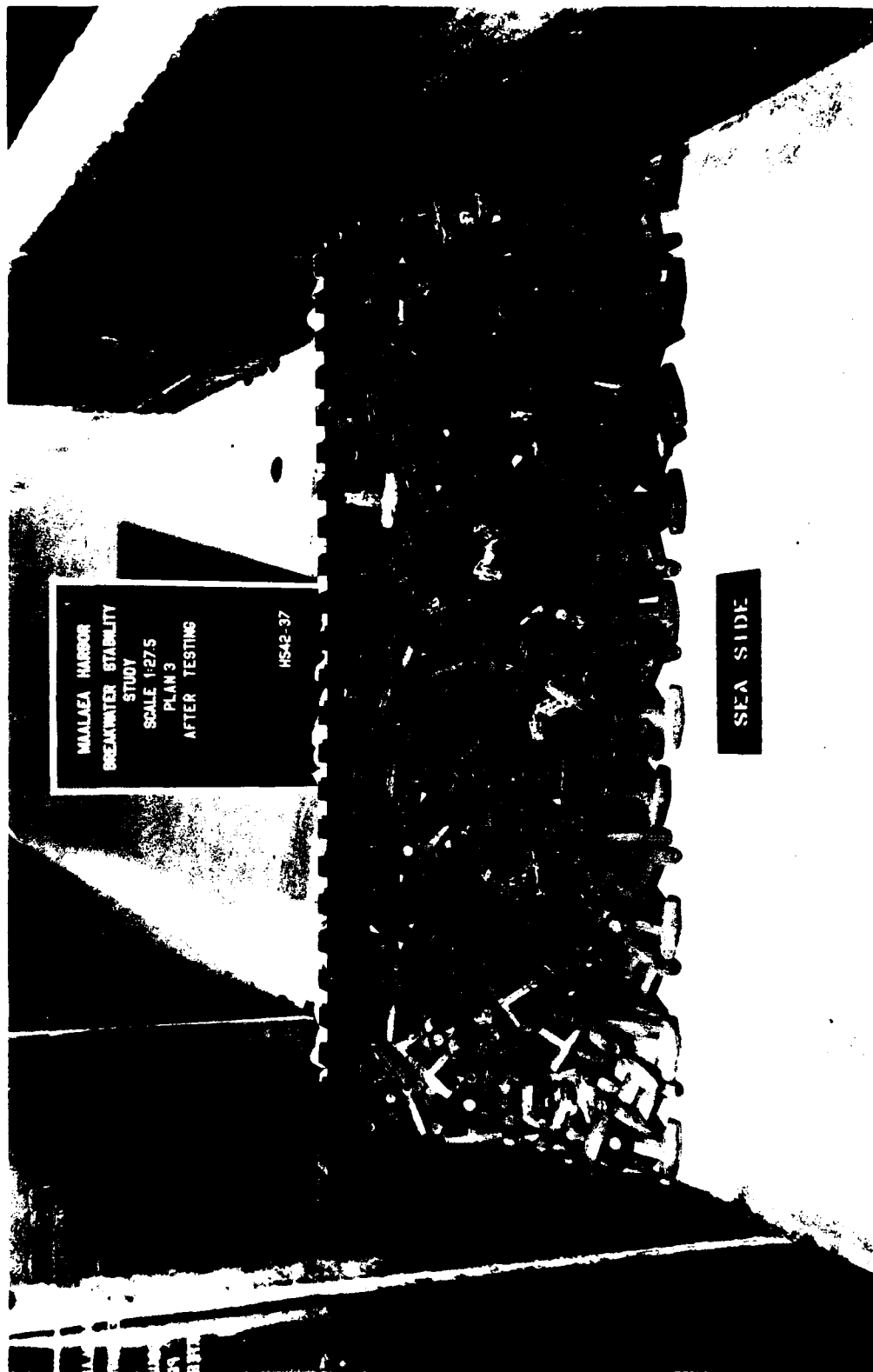


Photo 28. General view of plan 3 after testing of breakwater, showing condition after testing.

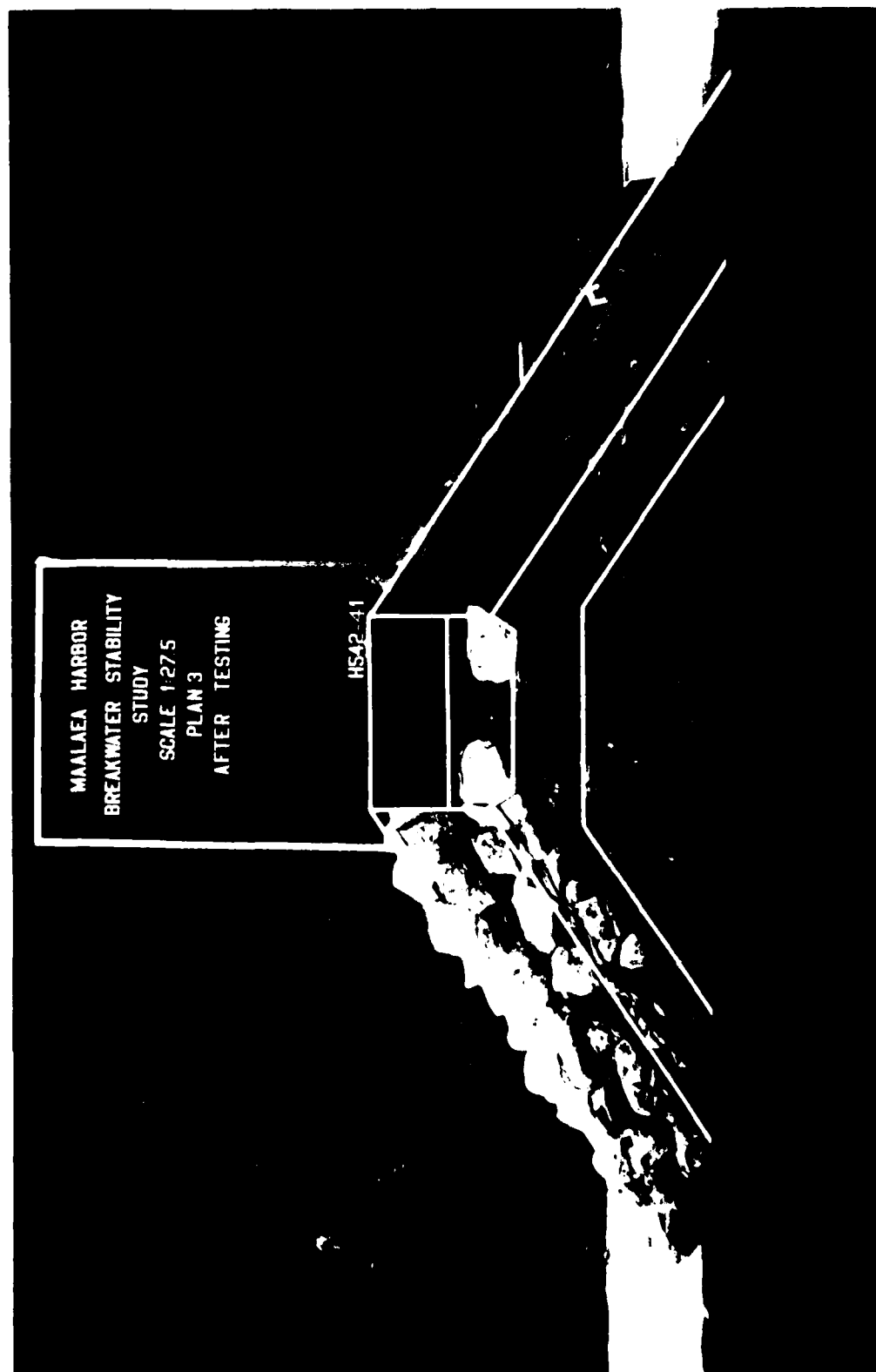


MAALEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:275
PLAN 3
AFTER TESTING

HS42-36

BEACH SIDE

Photo 29. Beachside view of film 3 after attack of 1000 ft. wave, 11:15 a.m., 11/10/64.



MAALAE HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:275
PLAN 3
AFTER TESTING

HS42-41

Photo 23. Side view of Plan 3 after attack of 16-sec, 16.7-ft breaking wave at an angle of 45° to mlt.

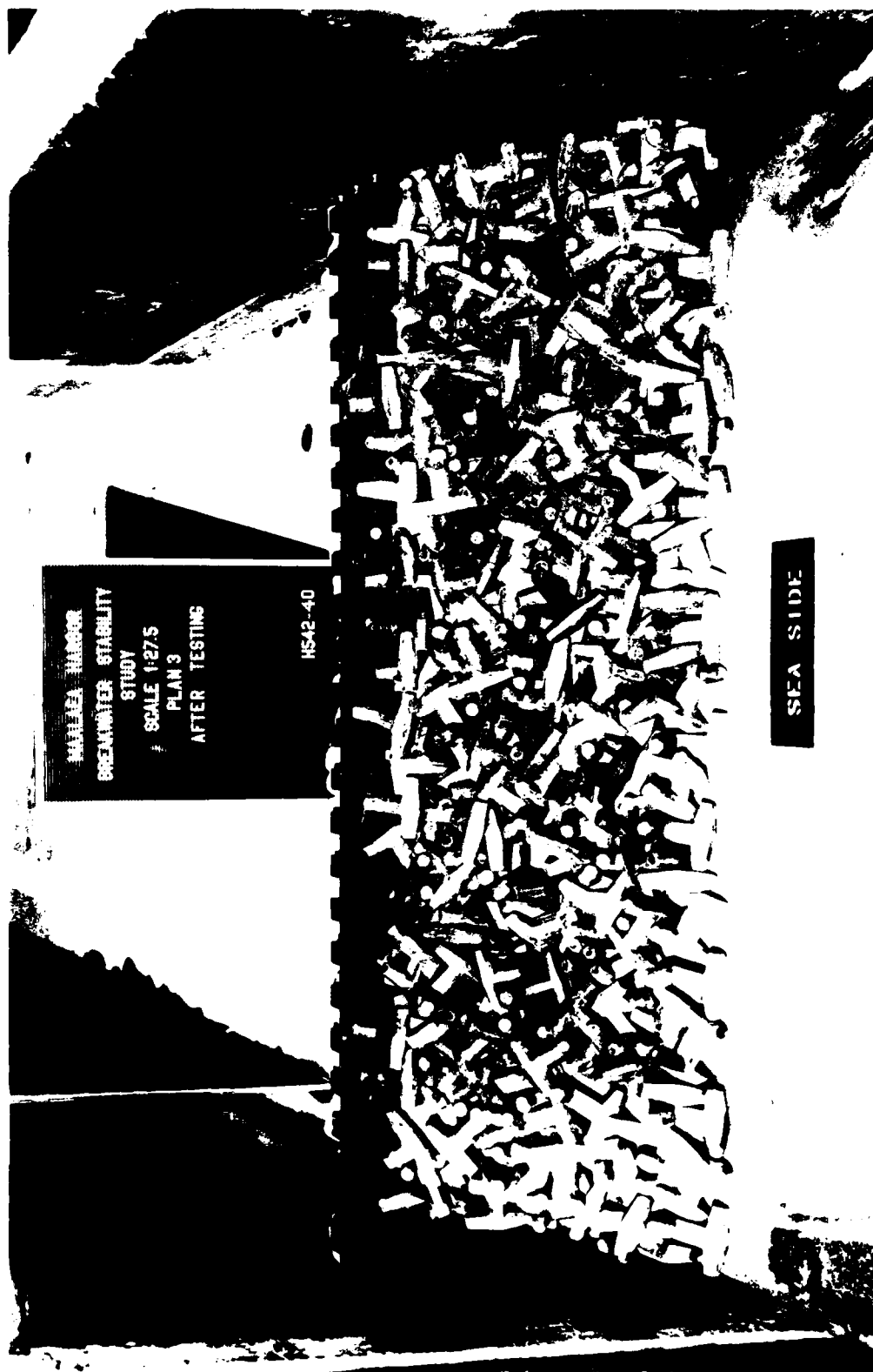


Photo 21. Sea-side view of Plan 3 after attack of 16-sec, 16.7-ft breaking waves at an angle of 45° to pile

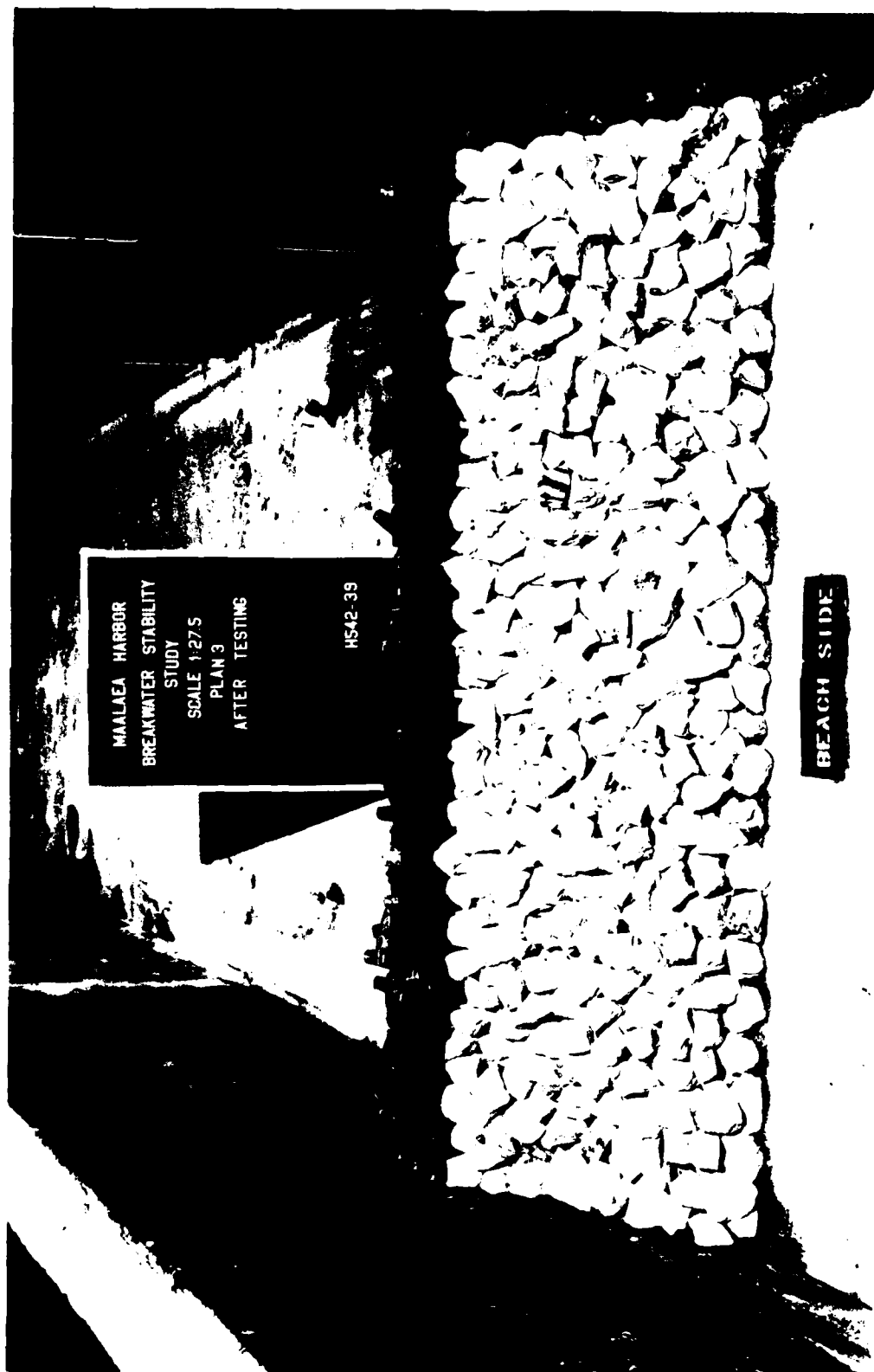


Photo 32. Beach-side view of Plan 3 after attack of 16-sec, 16.7-ft breaking waves at an SWL of +3 ft, 111w

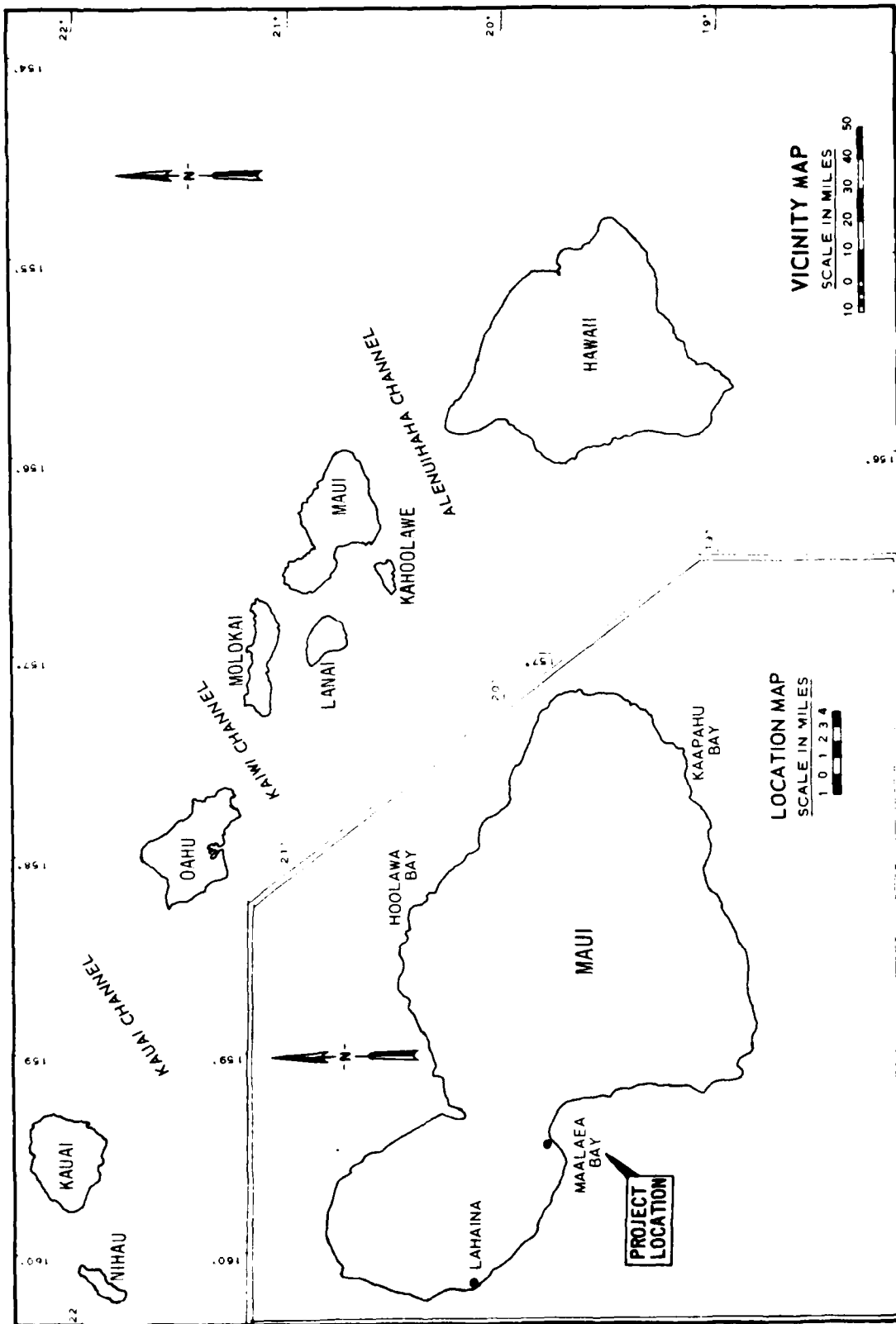


PLATE 1

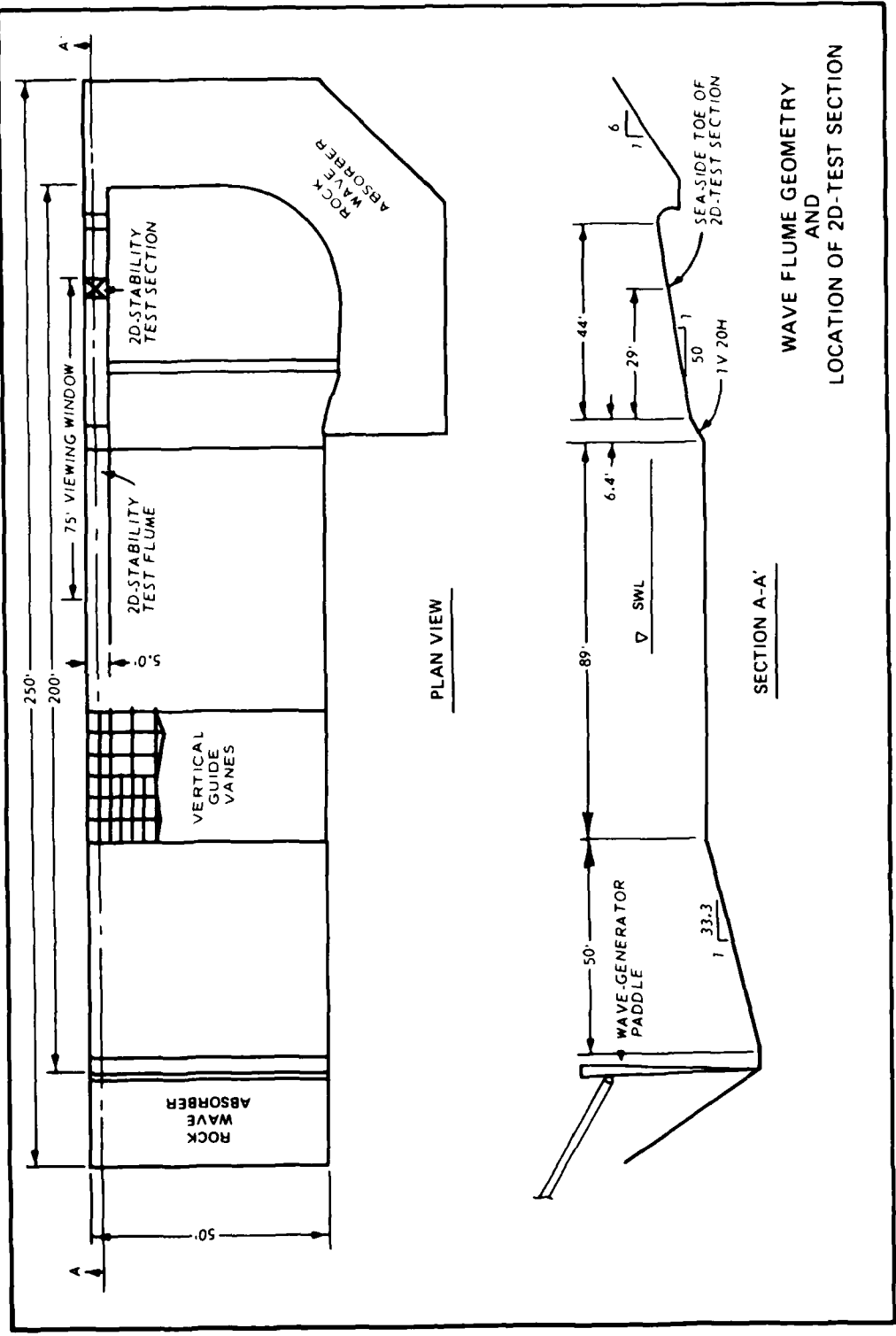
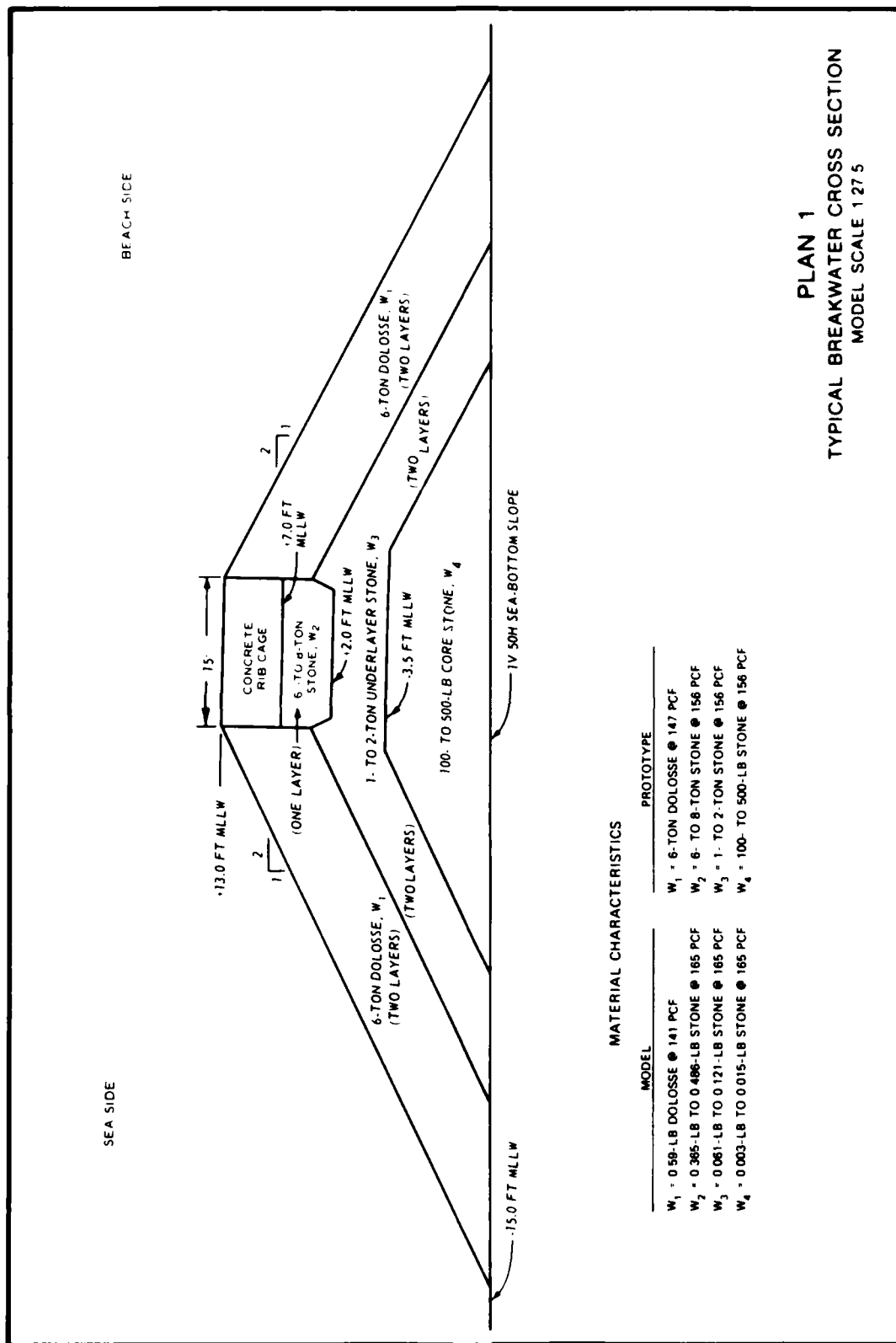


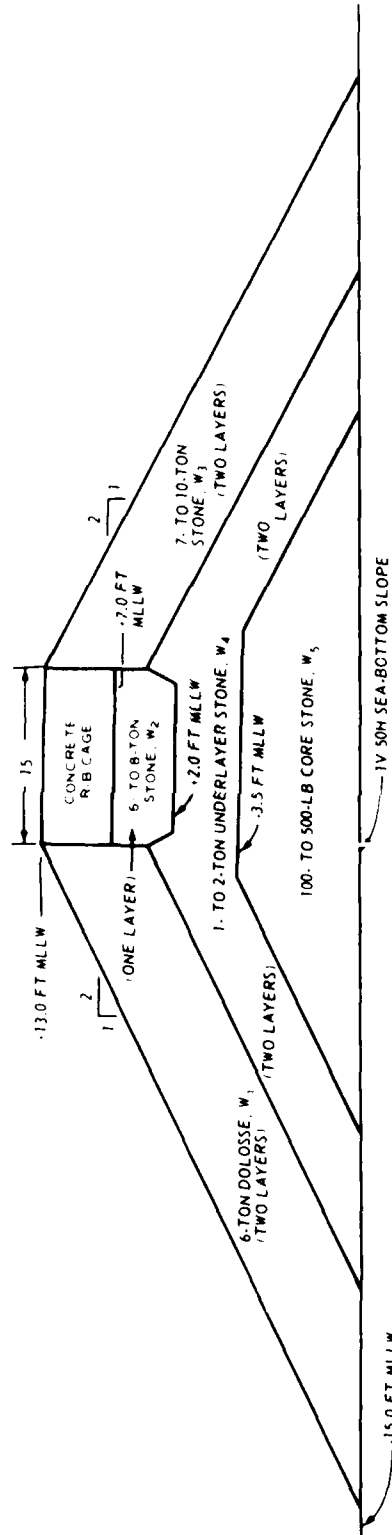
PLATE 2



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W ₁ = 0.59-LB DOLOSSE @ 141 PCF	W ₁ = 6-TON DOLOSSE @ 147 PCF
W ₂ = 0.365-LB TO 0.486-LB STONE @ 165 PCF	W ₂ = 6- TO 8-TON STONE @ 156 PCF
W ₃ = 0.061-LB TO 0.121-LB STONE @ 165 PCF	W ₃ = 1- TO 2-TON STONE @ 156 PCF
W ₄ = 0.003-LB TO 0.015-LB STONE @ 165 PCF	W ₄ = 100- TO 500-LB STONE @ 156 PCF

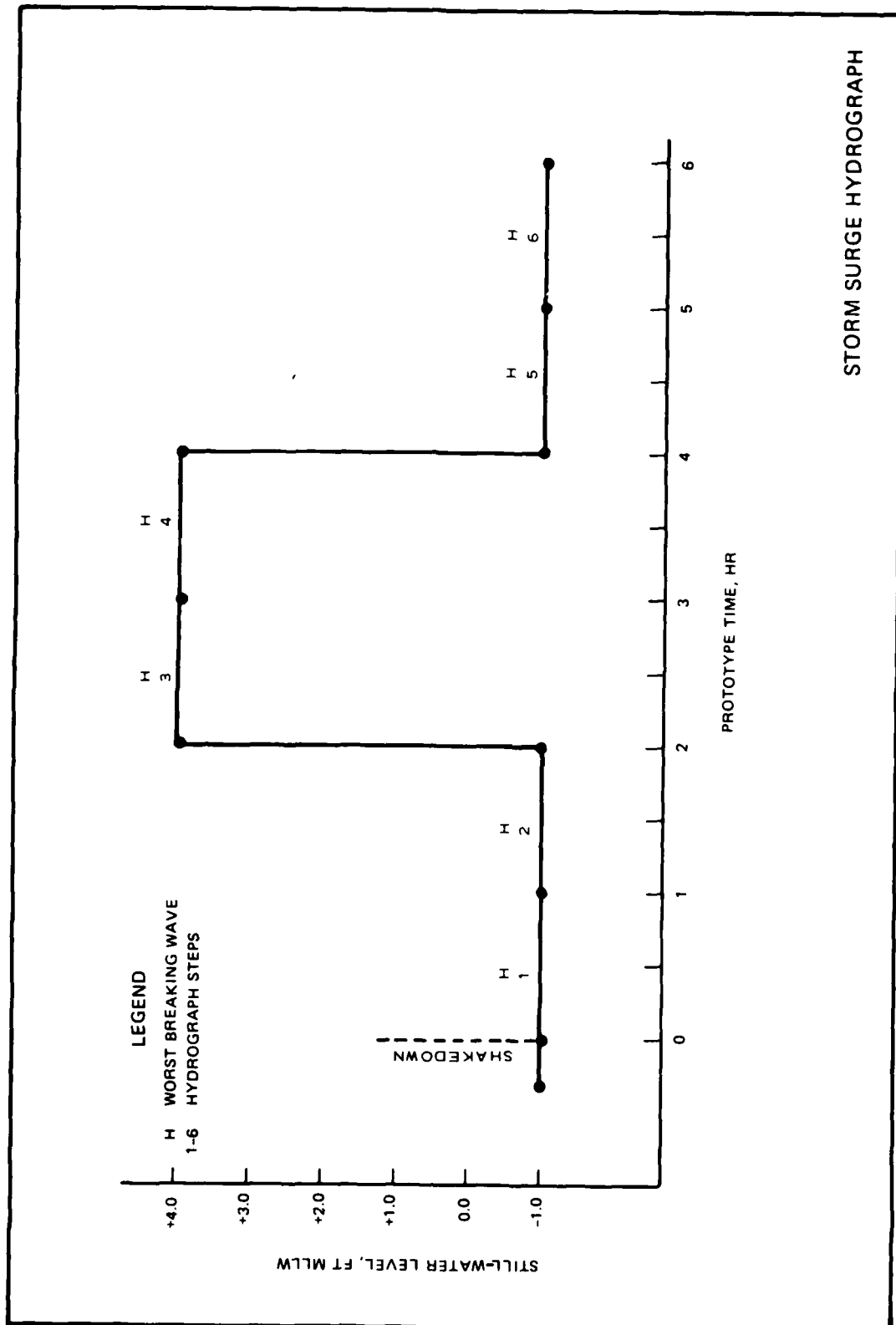
PLAN 1
TYPICAL BREAKWATER CROSS SECTION
MODEL SCALE 1/27.5



PLAN 2
TYPICAL BREAKWATER CROSS SECTION
MODEL SCALE 1:27.5

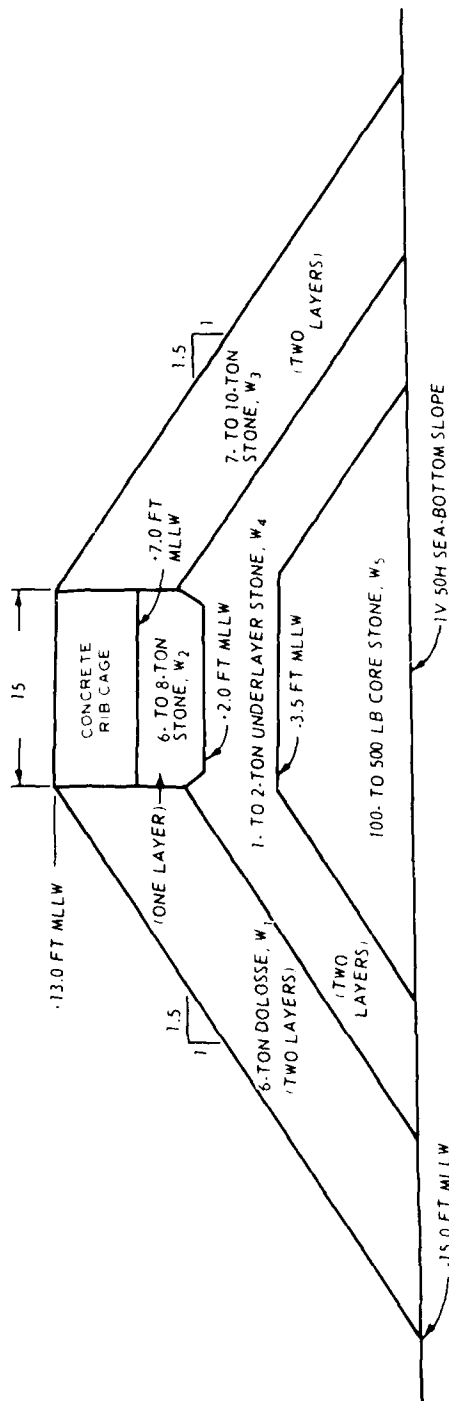
MATERIAL CHARACTERISTICS

MODEL		PROTOTYPE	
W_1	= 0.59-LB DOLOSSE @ 141 PCF	W_1	= 6-TON DOLOSSE @ 147 PCF
W_2	= 0.365-LB TO 0.486-LB STONE @ 165 PCF	W_2	= 6- TO 8-TON STONE @ 156 PCF
W_3	= 0.426-LB TO 0.608-LB STONE @ 165 PCF	W_3	= 7- TO 10-TON STONE @ 156 PCF
W_4	= 0.061-LB TO 0.121-LB STONE @ 165 PCF	W_4	= 1- TO 2-TON STONE @ 156 PCF
W_5	= 0.003-LB TO 0.015-LB STONE @ 165 PCF	W_5	= 100- TO 500-LB STONE @ 156 PCF



SEA SIDE

BEACH SIDE



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
w_1 : 0.59-LB DOLOSSE @ 141 PCF	w_1 : 6-TON DOLOSSE @ 147 PCF
w_2 : 0.365-LB TO 0.486-LB STONE @ 165 PCF	w_2 : 6- TO 8 TON STONE @ 156 PCF
w_3 : 0.426-LB TO 0.608-LB STONE @ 165 PCF	w_3 : 7- TO 10 TON STONE @ 156 PCF
w_4 : 0.061-LB TO 0.121-LB STONE @ 165 PCF	w_4 : 1 TO 2-TON STONE @ 156 PCF
w_5 : 0.003-LB TO 0.015-LB STONE @ 165 PCF	w_5 : 100- TO 500-LB STONE @ 156 PCF

PLAN 3
TYPICAL BREAKWATER CROSS SECTION
MODEL SCALE 1:27.5

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Carver, Robert D

Stability of rubble-mound breakwater Maalaea Harbor, Maui, Hawaii; hydraulic model investigation / by Robert D. Carver, Dennis G. Markle. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1981.

12, [32] p. [6] leaves of plates : ill. ; 27 cm.
(Miscellaneous paper - U. S. Army Engineer Waterways Experiment Station ; HL-81-1)

Prepared for U. S. Army Engineer Division, Pacific Ocean, Fort Shafter, Hawaii.

1. Breakers (Water waves). 2. Breakwaters. 3. Hydraulic models. 4. Maalaea Harbor, Hawaii. 5. Rubble-mound breakwaters. I. Markle, Dennis G., joint author. II. United States. Army. Corps of Engineers. Pacific Ocean Division. III. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper ; HL-81-1.
TA7.W34m no.HL-81-1

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